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Handbook of Advanced Treatment Review Issues

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OF
ADVANCED TREATMENT REVIEW ISSUES

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This handbook identifies the most significant issues encountered in the review of proposed advanced treatment (AT) projects by the Environmental Protection Agency (EPA) Headquarter's AT Task Force, and describes the procedures and information used to evaluate each issue as a basis for making Federal Construction grant funding decisions.

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Table of Contents

Chapter No.		Page No.
1.0	Introduction	1-1
1.1	Purpose	1-1
1.2	Background and Scope	1-2
1.3	Summary of Technical Issues	1-4
2.0	Treatment Process Issues	2-1
2.1	Design Wastewater Flow and Pollutant Loading	2-1
2.2	Effects of Wastewater Temperature on the Cost of Nitrification	2-4
2.3	CBOD vs BOD	2-11
2.4	Costs and Pollutant Reduction Capability of Nitrification and Tertiary Filtration	2-12
2.5	DO Improvement Provided by Tertiary Filtration	2-15
2.6	Tertiary Filtration to Improve Reliability	2-16
2.7	Tertiary Filtration to Remove Suspended Solids	2-17
2.8	Upgrading Trickling Filters to Achieve Nitrification	2-18
2.9	Use of Flocculation Channels for Reducing CBOD and SS from Trickling Filters	2-20
2.10	Oxidation Ditches	2-22
2.11	Post-Aeration	2-23
3.0	Water Quality Issues -- Dissolved Oxygen	3-1
3.1	Dissolved Oxygen Criteria	3-2
3.2	Reaction Rates	3-3
3.2.1	Carbonaceous Deoxygenation Rate (Kd)	3-3
3.2.2	Nitrogenous Deoxygenation Rate (Kn)	3-6
3.2.3	Sediment Oxygen Demand (SOD) and Reaction Rate Effects	3-9
3.2.4	Reaeration Rate (Ka)	3-11
3.2.5	Temperature Effects on Reaction Rates	3-13
3.2.6	Background CBOD and Reaction Rate Effects	3-14
3.3	Selecting Seasonal Conditions for Wasteload Allocations	
3.3.1	Flow Selection	3-16
3.3.2	Seasonal Water Quality Criteria	3-16
3.3.3	Receiving Water Temperature Selection	3-17
3.3.4	Wasteload Selection	3-18
3.3.5	Seasonal Wasteload Allocation Analyses	3-19
3.4	Pollutant Loadings	
3.4.1	$CBOD_u$ Loading ($CBOD_u$:$CBOD_5$ Ratio)	3-24
3.4.2	Calculation of NOD_u Loadings	3-26
3.5	Reaction Rate Adjustment (Model Calibration)	3-27
3.6	Permit Averaging Periods	3-29

Glossary

WLA - wasteload allocation

BOD$_5$ - biochemical oxygen demand - 5 day

CBOD$_5$ - carbonaceous biochemical oxygen demand - 5 day.

CBOD$_5$ - ultimate carbonaceous biochemical oxygen demand - 5 day.

NOD$_u$ - ultimate nitrogenous oxygen demand

7Q10- - once in ten year, seven day average flow

Kn - nitrogenous decay rate (per day)

Kd - carbonaceous decay rate (per day)

Ka - reaeration rate (per day)

SOD - sediment oxygen demand (gm/m^2/day)

P-R - photosynthesis minus respiration.

CSO - combined sewer overflow

MGD - million gallons per day

mg/l - milligram per liter

TRC - total residual chlorine

1.0 Introduction

1.1 Purpose

This handbook identifies the most significant issues encountered in the review of proposed advanced treatment (AT) projects by the Environmental Protection Agency (EPA) Headquarter's AT Task Force, and describes the procedures and information used to evaluate each issue as a basis for making Federal construction grant funding decisions. The information contained in this document is intended to assist States and Regions in the planning and review of AT projects. It does not supersede or replace State requirements or procedures related to advanced treatment. States may require AT facilities to achieve more stringent treatment than those found justified for EPA funding under the Agency's AT review policy.

This handbook supplements, but does not substitute for, the Agency's AT policy[1], the technical appendix to the policy[1] and related guidance documents on water quality standards, wasteload allocations, permitting and treatment process design. Technical guidance documents available from EPA and other sources are referenced herein as appropriate to avoid duplication. Users of this document are presumed to have basic knowledge of water quality analysis and modelling procedures as well as treatment facility design practices.

The purpose of the AT review process is to ensure that any Federally funded AT processes will definitely result in significant water quality and public health improvements as required by the AT Congressional directive.

Accordingly, when AT processes lack technical support at the time of the AT review, their funding is deferred even though some of these processes may subsequently be technically justified and funded as necessary to attain designated water uses. The handbook identifies these technical uncertainties and encourages the collection of addition data and analyses, and thus will improve the technical justification for future AT processes.

1.2 Background and Scope

In October 1978, the Appropriations Conference Committee of the U.S. Congress directed the Environmental Protection Agency (EPA) Headquarters to review advanced treatment (AT) projects with incremental costs beyond secondary greater than $1 million; and, as a prerequisite for Federal funding, that the Administrator personally determine that such projects will "definitely result in significant water quality and public health improvements." In 1980, the Appropriations Conference Committee raised the cost threshold for AT review to $3 million.

In March 1979, EPA issued the first AT review policy[2], Program Requirements Memorandum (PRM) 79-7, to implement the Congressional AT Directive. Under terms of a Settlement Agreement reached with the State of Illinois as a result of a suit brought by the State contesting the AT review, on June 20, 1980 EPA published a revised draft AT policy. On April 26, 1984, the Assistant Administrator for Water signed a new AT review policy[1] incorporating many of the comments from States and EPA Regional Offices on the published draft policy and subsequent drafts.

As of December 1983, EPA Headquarters had completed reviews of 77 proposed AT projects. The following table summarizes the AT Task Force funding recommendations approved by the Administrator for these 77 projects. A detailed summary is also presented in tabular form in Appendix A.

Process	Numbers of AT Processes Reviewed*		
	Proposed	Approved	Deferred
Activated Sludge or Trickling Filter**	65	58	7
Tertiary Filtration	54	21	33
Dechlorination***	11	10	1
Winter Ammonia Removal**	9	0	9
Breakpoint Chlorination**	4	3	1
Land Treatment	4	4	0
Denitrification	3	1	2
Oxidation Ditch**	2	2	0
Interbasin Transfer	2	2	0
RBCs	1	1	0
Carbon Absorption	1	1	0

* Number of processes exceeds the total number of projects because most projects include more than one AT process; e.g., tertiary filtration was usually proposed in conjunction with ammonia removal.

** Ammonia removal processes

*** Disinfection processes and dechlorination are not AT. However, the need for dechlorination is evaluated in AT reviews.

As shown in Appendix A and the above table, most of the 77 AT projects reviewed included processes for biological ammonia removal (nitrification). AT facilities for ammonia removal were approved for Federal funding in 63 (88 percent) of the 71 proposed projects that included ammonia removal. Tertiary filtration was approved for funding in 21 (39 percent) of the 54 projects that included filtration. Land treatment, dechlorination, oxidation ditches and projects involving interbasin transfer in lieu of AT were approved in most cases. On the other hand, winter ammonia removal requirements and denitrification were deferred in most cases.

This document is a compilation of the technical issues evaluated in the 77 AT reviews completed by the AT Task Force between 1978 and 1983. The issues discussed are taken from all aspects of the planning process including water quality standards, water quality analyses, permits, facility planning and AT process design. Issues related to the control of CBOD and ammonia to meet dissolved oxygen criteria and the control of ammonia and chlorine to prevent acute and chronic toxicity are included in this document. Issues evaluated in the review of AT projects involving the control of nitrogen and phosphorus are not included in this document due to the experimental status of many of the nutrient control technologies and the complexity of nutrient related water quality problems and analytical procedures.

1.3 Summary of Technical Issues

A summary follows of the major technical issues raised by Headquarters AT reviews and discussed in this document follows. Each issue is referenced to the report section where the discussion appears.

Treatment Process Issues -- Chapter 2.0

o Appropriate waste loading and wastewater flow during the critical summer period -- For some AT projects, infiltration/inflow comprised more than 75 percent of design flow capacity for the treatment facility. In one case, the treatment capacity necessary to meet secondary treatment requirements during the wet weather flow period was found to provide sufficient capacity to meet AT (nitrification) requirements during the summer months. In another case with high I/I, the treatment processes were designed based on a waste loading that was calculated using the dry weather influent concentration and the wet weather flow. In these situations, the proposed AT design was based on a waste loading that would not likely occur, and resulted in AT processes or capacity beyond those needed to meet water quality standards. See Section 2.1.

- Effects of wastewater temperature on facility design -- Since wastewater temperature affects AT process selection and facility design, inadequate temperature analysis can result in unnecessary AT processes or capacity. In some cases, conditions for non-critical cooler months governed the design of the treatment facility. This resulted in AT treatment facilities capacity beyond that supported by the WLA conducted for critical conditions (e.g., low stream flow, high temperature). See Sections 2.2.

- Use of BOD data instead of CBOD data to characterize treatment process performance -- For most of the AT projects reviewed, performance of treatment facilities was generally based on BOD rather than CBOD data. Since BOD test data during warm weather often reflect oxidation of ammonia as well as carbonaceous oxygen demand, the performance of secondary and AT facilities has often been underestimated. See Section 2.3.

- Sufficiency of data on the performance and DO improvement provided by tertiary filtration as an add-on to nitrification facilities -- Tertiary filtration affords only limited pollutant removal relative to its cost. Therefore, site-specific data and careful analyses are required to accurately assess the water quality improvement from tertiary filtration. See Sections 2.4, 2.5 and 2.6.

- Advanced treatment capability of modified trickling filters -- Some AT projects for ammonia removal proposed replacement of existing trickling filters with activated sludge units. Rehabilitation of trickling filters with plastic media or flocculation channels was found in some cases to be more cost-effective than the proposed activated sludge replacement. See Sections 2.8 and 2.9.

- DO improvement provided by post-aeration -- In some AT projects that were located on effluent dominated streams, post-aeration was not included as part of the proposed project and would have resulted in a significant improvement in the DO in the receiving stream. Post-aeration can result in a significant improvement in the DO at the mixing point and at the critical DO sag point downstream, and may be more cost-effective than removal of CBOD or ammonia. See Section 2.11.

Water Quality Issues -- Dissolved Oxygen -- Chapter 3.0

- The effect of algae and other aquatic plants on rate constants -- Many receiving waters had some algal growth either suspended or attached that was not considered in the determination of rate constants. In particular, the Kn rate can be over-estimated when determined solely based on the loss of ammonia if large amounts of algae are present. The loss of ammonia can be caused by the algal uptake of the ammonia rather than oxidation of ammonia by nitrifying bacteria. In addition, if suspended algal are present in the BOD test bottle, algal respiration and decay can result in erroneous BOD measurements and over estimation of the Kd rate. These algal effects on decay rates can result in over-estimation of the DO improvement from nitrogenous and carbonaceous BOD removal. See Sections 3.2.1 and 3.2.2.

- Consideration of the effects of sediment oxygen demand -- Sediment oxygen demand (SOD) was rarely included in mathematical models. To calibrate the model to actual in-stream DO data, the effect of SOD was incorporated into other modeling rates such as Kd, Kn and Ka. In cases where Ka was determined using a reaeration formula and not adjusted, the oxygen demand from SOD was incorporated into the Kd or Kn rates. This had the effect of over-estimating the DO impact from removal of nitrogenous or carbonaceous BOD. See Sections 3.2.3 and 3.5.

- Reliance on empirically derived formulae in the selection of reaeration rates -- The reaeration rate (Ka) was rarely measured as part of the development of a mathematical model. In addition, where empirical formulae were used, frequently no analysis was performed to evaluate the uncertainty of model results associated with Ka. In some cases, particularly in streams with very low velocities, AT requirements were established primarily because of a low Ka rate calculated using an empirical formulae. Where adoption of a low Ka rate would result in an expensive AT process, site specific measurement of Ka would significantly reduce model uncertainty. See Section 3.2.4.

- Selection of stream temperature and flow for DO analyses -- Temperature has a significant effect on decay rates and modeling results. In some cases, maximum temperatures were used in modeling rather than average monthly temperatures. The monthly average temperature may be more appropriate when modeling chronic effects; i.e. exposure for 30-days. Also, the lowest flow condition was often matched with the highest temperature condition without determining whether these conditions would occur simultaneously. This could result in a critical temperature and flow condition that together would have an extremely small probability of occurrence. See Sections 3.2.5 and 3.3.

- Seasonal water quality criteria -- Stringent DO criteria for coldwater fisheries have been used in some cases even though coldwater species do not inhabit the receiving water during the critical summer low flow period. In these instances, the model used to justify the AT requirements was run at temperatures not conducive to coldwater fish; e.g. 25°C to 30°C. In cases where coldwater fishery uses exist only during the non-summer seasons, seasonal use designations and criteria may be needed to prevent unnecessary treatment requirements. See Section 3.3.2.

- Selection of wasteloads -- In some cases, waste loadings used in modeling were determined based on the annual average effluent quality (e.g. annual permit limitations) instead of the actual effluent flows and concentrations that would likely occur during the critical summer period. This was particularly important where annual rather than seasonal effluent limits are used, since an annual limit represents the performance that the facility must achieve during the coldest or wettest month of the year. In most cases, a facility that is providing secondary treatment during the winter period can achieve a certain level of AT during the summer period. Therefore, the pollutant loadings input to the model should reflect the seasonal variations in treatment performance. See Section 3.3.4.

- Seasonal wasteload allocation analyses to justify seasonal effluent limitations -- In general, DO modeling was performed only during the critical summer period. No additional modeling was usually done to determine whether the level of treatment specified by the model was needed to meet water quality standards during the period covered by the permit. Since more treatment capacity is needed at lower temperatures due to reduced reaction rates, lack of a seasonal WLA analyses can result in greater treatment requirements and costs than is necessary to meet water quality standards. See Section 3.3.5.

- Determination of the NODu loading -- The expected total Kjeldahl nitrogen (TKN) concentration has been used to calculate the NOD loading for the mathematical model without evaluating the impact of the organic nitrogen in the stream. The organic nitrogen may not hydrolize to become ammonia and create a DO demand within the critical segments of the stream. Using TKN rather than ammonia can result in over-estimation of the DO impact of the nitrogen component of the wastewater. See Section 3.4.1.

- Adjustment of $CBOD_u$:$CBOD_5$ ratios to reflect different types of wastewater -- In some cases, a constant $CBOD_u$:$CBOD_5$ ratio was used to calculate pollutant loadings for various levels of treatment and types of wastewaters. However, this ratio will vary as the level of treatment increases and can vary for different wastewaters, particularly those that include an industrial component. See Section 3.4.2.

- Adjustment of reaction rates -- In many cases, reaction rates determined in calibrations were not adjusted in projection model runs to reflect how these rates will change with higher treatment levels. Although there is a lack of information in the literature on exactly how decay rates change as treatment levels change, the carbonaceous decay rate will generally decrease at higher levels of treatment due to the greater biological stability of the treated effluent. Ignoring this factor could result in more AT than necessary, particularly if rates used to evaluate AT were determined from stream data when the facility was achieving a lower level of treatment. See Section 3.5.

- Establishment of effluent limitation averaging periods -- In some cases, effluent limitations were written to meet chronic criteria but expressed as daily maximum concentrations. A daily maximum limitation requires greater treatment plant reliability and capacity than is justified by a WLA analysis using chronic exposure (30 day) criteria. See Section 3.6.

Water Quality Issues -- Ammonia and Chlorine -- Chapter 4.0

- Importance of pH and temperature data in the evaluation of ammonia toxicity -- Very few projects were proposed solely to prevent a water quality problem related to ammonia toxicity. However, in those cases where it was proposed, very limited pH and temperature data were available with which to make an accurate estimate of toxic effects from un-ionized ammonia. Because of the sensitivity of ammonia toxicity to pH and temperature and the relatively large cost associated with biological ammonia removal, especially during colder temperatures, a data base on temperature and pH is needed. See Chapter 4.1

- Consideration of chlorine toxicity where chlorination was proposed -- Dechlorination was not proposed where mass balance calculations of instream chlorine concentrations showed that acute and chronic toxic effects would likely occur. See Chapter 4.2.

2.0 Treatment Process Issues

This section discusses the facilities planning issues affecting the water quality based justification of AT processes reviewed by the AT Task Force. Each of the subsections below evaluates an issue that arose in at least one AT review. The issues discussed in this section fall into three general categories: (1) seasonal and temperature variable effects on wastewater flow and AT process performance, (2) the cost and pollutant removal capabilities of commonly proposed AT processes and (3) upgrading of existing secondary treatment facilities to achieve advanced treatment levels. The reader is referenced to U.S. EPA documents on treatment process design[3,4,5,6,7].

2.1 Design Wastewater Flow and Pollutant Loading

The design wastewater flows and pollutant loadings obviously affect the design and the cost of wastewater treatment facilities. What may not be as obvious is how seasonal variations in flows and loadings can effect the design of secondary and advanced treatment processes.

Seasonal variations in wastewater flows and loadings can result from infiltration/inflow (I/I) in the sewer system, seasonal or intermittent discharges of industrial wastewaters or seasonal fluctuations in population such as in academic or tourist communities. The major issues related to wastewater flows and loadings encountered in AT reviews involved sewer systems that contained a significant amount of I/I; i.e., up to 75 percent of the design capacity. In such cases, either the wastewater flow or pollutant concentrations used to calculate the design pollutant loadings were not

not appropriate for the critical period under evaluation in the modeling analysis.

Overestimation of the pollutant loadings used to design treatment processes occurred in many AT projects reviewed by the AT Task Force. In one instance, the pollutant loadings for BOD_5 and ammonia used to design the treatment processes assumed the maximum 30-day average flow period (high I/I) and the dry weather influent concentration. For example, during dry weather periods the influent $CBOD_5$ loading is 20,850 pounds per day (lbs/day) at an influent $CBOD_5$ concentration of 250 mg/l and a flow of 10 MGD. During wet weather periods at 20 MGD, the loading would be considerably less than twice this loading since the increase in flow is due to I/I. Waste loadings that reach the plant should not change proportionally with flow fluctuations caused by I/I. While the loadings at 20 MGD may be greater than 20,850 lbs/day due to scouring of the sewer system, domestic wastewater in combined sewer systems or cross connections in separated sewer systems, the loading would likely be considerably less than twice the dry weather loading. Therefore, the pollutant loading used to design the facility during wet weather period will generally be overestimated when determined from the dry weather concentration and the wet weather flow.

In another instance, the pollutant loading used to design the AT facility during the summer low flow period was determined using the dry weather concentration and the wet weather maximum flow. The facility was overdesigned because the maximum monthly average flow would not likely occur during the critical low stream flow period. For example, assume an AT facility has requirements for ammonia removal from June through September and secondary treatment during the remainder of the year. The facility could be designed to provide only secondary treatment for the maximum monthly average flow occurring from October to June and to remove ammonia for the maximum monthly average flow from June through September.

One AT review illustrating the seasonal design flow issue featured an AT project with proposed facilities for seasonal nitrification. In this case, I/I studies and flow projections indicated that I/I comprised 75 percent of the annual maximum monthly average flow. The issue concerned whether additional aeration capacity was needed to achieve nitrification during low stream flow periods in the summer. The AT Task Force concluded that the aeration capacity proposed for achieving secondary treatment during wet weather periods also afforded sufficient capacity to achieve nitrification for the critical low stream flows during the summer.

The AT Task Force evaluated whether I/I during summer wet weather periods might increase wastewater flow enough to prevent nitrification. This would violate summer AT effluent limitations. The Task Force determined that higher stream flows would occur simultaneously with higher wastewater flows caused by I/I. Therefore, during those periods, exceedence of water quality standards would not likely occur even though the monthly average ammonia limits of the permit were temporarily not met. One approach for avoiding such noncompliance involves permit coverage at a wastewater flow (AT limits apply at less than the prescribed flow cut-off) as well as a time period (AT limits apply during prescribed dates). On this basis, when wastewater flow exceeds the flow cut-off, only the secondary treatment limitations would apply; even during the time period when AT limits are in effect.

2.2 Effect of Wastewater Temperature on the Cost of Nitrification

An evaluation of the cost of nitrification facilities shows their capital and operating costs are highly dependent on the wastewater temperature. In turn, the wastewater temperature used as the basis of design depends on when nitrification is required to prevent exceedence instream of DO and ammonia criteria. Accordingly, the period when nitrification is required will greatly affect costs of nitrification facilities.

Two processes, activated sludge and trickling filter systems, were most often proposed to achieve nitrification during warm weather months. In some cases, wastewater temperature was not adequately considered in setting the ammonia effluent limitations for the project. In fact, year-round nitrification was proposed for several AT projects based only on summer wasteload allocations. In other less obvious cases, AT effluent limitations requiring ammonia removal were established for months when wastewater temperatures would be significantly less than would occur during the critical period modeled in the wasteload allocation analysis. These practices led to proposals for unnecessary nitrification capacity included in some of the AT projects reviewed.

The capital and operation and maintenance (O&M) cost estimates shown in Tables 2-1 and 2-2 were used to develop the present worth cost estimates for the 10 MGD facilities indicated on Figures 2-1 and 2-2. Cost estimates for nitrification at 20°C shown in Tables 2-1 and 2-2 and Figures 2-1 and 2-2 were taken from the Innovative and Alternative Technology Assessment Manual[3].

Cost estimates for nitrification at wastewater temperature other than 20°C were estimated using standard engineering practice, i.e., design parameters for each process type were adjusted to account for the effect of temperature on the biological treatment processes. Although these estimates depend partly on site-specific conditions, the estimates nevertheless indicate general trends in nitrification costs as a function of wastewater temperature. These cost estimates reflect only the cost to upgrade existing secondary facilities to provide nitrification, and do not include any costs for expansion of existing primary or secondary facilities.

The effects of wastewater temperature on costs were evaluated for the following methods of upgrading an activated sludge system to provide nitrification:

Case 1 - Combined carbonaceous BOD reduction and nitrification in a single stage aeration system at conventional (2500 mg/l) mixed liquor suspended solids (MLSS) levels; nitrification accomplished by providing additional aeration and/or additional aeration time.

Case 2 - Combined carbonaceous BOD reduction and nitrification is a single stage aeration system at a raised (3750 mg/l) MLSS level. Includes additional clarification.

Case 3 - Separate second stage nitrification system following first stage carbonaceous BOD removal, assuming a MLSS of 2000, first stage effluent BOD of 50 mg/l and ammonia of 20 mg/l.

Also evaluated were the following approaches for upgrading a trickling filter system to provide nitrification:

Case 1 - Replacement of rock media with plastic media to provide approximately twice the available surface area. This alternative will achieve nitrification at wastewater temperatures generally greater than 20°C.

Case 2 - Case 1 plus chemical pretreatment utilizing the primary clarifiers for sludge removal. This is assumed to reduce the load to the trickling filter by 50 percent and provide nitrification at temperatures above 15°C.

Case 3 - Addition of first stage plastic media trickling filters to reduce loading by 50 percent and achieve nitrification at 15°C. Achieving nitrification at lower temperatures in a single stage system was assumed to be non cost-effective.

Case 4 - Second stage plastic media trickling filter system to achieve nitrification at year-around temperatures of 5°C or higher.

Case 5 - Second stage activated sludge system following the first stage trickling filter, similar to activated sludge; Case 3.

For all estimates, the percent ammonia reduction was assumed to be the same year-round. If less stringent levels of nitrification were required during cold weather, the cost estimates for colder wastewater temperatures would be reduced by an amount dependent on the difference in the percent ammonia removal required during the warm weather and cold weather. If the percentage ammonia reduction during warm weather approximates that during cold weather, the cost estimates for these two conditions would be similar to the estimates presented in Tables 2-1 and 2-2, and Figures 2-1 and 2-2. In general, these cost estimates would apply if design effluent ammonia concentrations at temperatures of 5-10°C were less than double those at 20°C (e.g. 2.5 mg/l summer, 4.0 mg/l winter). If the design ammonia concentration at temperatures of 5-10°C was more than double that at 20°C, the capital cost for nitrification at lower wastewater temperatures could be lower than the estimates presented. Case-by-case analyses would be necessary to accurately determine the exact effects on costs of different seasonal ammonia limitations.

As indicated on Table 2-1, the capital and O&M costs for all types of activated sludge nitrification systems vary significantly depending on the design wastewater temperature. For example, the capital cost to achieve nitrification using activated sludge at a wastewater temperature of 5°C would be nearly nine times more than the capital cost of achieving nitrification at 20°C (i.e., $6.2 million using Case 3 at 5°C versus $0.7 million using Case 1 at 20°C). Similarly, Table 2-2 shows significant cost differences for the trickling filter systems. Tables 2-1 and 2-2 also show significant differences in the annual O&M costs, depending on the design wastewater temperature, for all the nitrification cases analyzed.

The analysis of present worth costs of all the various cases (activated sludge and trickling filters) indicate trends similar to those described above. For example, Figure 2-1 indicates a present worth cost for Case 1 at temperatures above 20°C is about $1.3 million. The present worth cost of Case 3, the least costly approach, is about 9.6 million for design temperatures less than 10°C; seven times the cost at 20°C.

Figure 2-1 shows that the design wastewater temperature also affects relative costs of the various cases. For example, if nitrification is required when wastewater temperatures are 15-20°C, the present worth cost of two-stage nitrification (Case 3) is more than 1.5 times that of the single-stage nitrification options (Cases 1 and 2), which are about equal in cost. However, if nitrification must be achieved for wastewater temperatures of 5-10°C, the present worth cost for two-stage nitrification is significantly less than that for single-stage nitrification.

As Figure 2-2 shows, the present worth costs of trickling filter systems also vary significantly depending on the wastewater temperature. For example, if the design wastewater temperature is 20°C, nitrification can often be achieved without major construction by changing the filter media from rock to plastic (depending on the capacity of the existing system and the projected increase in wastewater flow). Figure 2-2 shows a present worth cost of about $2.5 million for Case 1 nitrification at a design temperatures of 20-25°C. With this approach, nitrification might not be achieved at wastewater temperatures of less than 15°C. Therefore, achieving nitrification when these lower wastewater temperatures occur would likely require either additional trickling filters or a second-stage activated sludge unit. If a second-stage activated sludge unit were added, the cost of Case 5 nitrification with temperatures of 10-15°C would be about $7.4 million, or nearly three times the cost the Case 1 nitrification for temperatures of 20-25°C.

Although actual cost estimates for all cases will vary depending on site-specific conditions, the generalized cost estimates presented herein clearly show the important impacts of design wastewater temperatures on the capital and operating costs for nitrification and on the relative costs of various process alternatives.

To illustrate the seasonal variability of wastewater temperature, Table 2-3 and Figure 2-3 show approximate estimates of monthly changes in wastewater temperature for selected States. The data in Figure 2-3 were taken from selected municipal treatment facilities in Michigan and Indiana. Table 2-3 values represent wastewater temperatures for more than 300 secondary treatment facilities throughout the U.S. collected for the development of the Secondary Treatment Regulation[8]. Table 2-3 values were developed from regression analyses of wastewater temperature versus ambient air temperatures based on air and wastewater temperature data from 40 facilities in New York State. This regression approach was necessary because sufficient wastewater temperature data were not available for facilites in States other than New York. The regression analysis produced an R^2 value of 0.71, which indicates a good correlation between wastewater temperature and ambient air temperature.

Review of Table 2-3 and Figure 2-3 suggests that seasonal differences in wastewater temperature can significantly affect nitrification costs, particularly in Northern States. For example, data from New York State indicate that wastewater temperatures are about 20°C or greater from July through September, and are 15°C or greater from June through October. Based on the generalized cost curves discussed previously, providing nitrification from July through September in New York would be much less costly than providing nitrification in June or October. Nitrification costs would increase even more sharply for November through May (temperature 15°C or below).

An examination of wastewater temperatures for more moderate climates indicates that temperature can markedly affect nitrification costs in these areas as well. For example, the data in Table 2-3 indicate that wastewater temperatures in North Carolina exceed 20°C from May through October. Thus, nitrification costs would be moderate for this six month period. Such nitrification might be accomplished with little additional expense, for example, by providing additional aeration tankage in an existing secondary activated sludge facility, or by changing the filter media from rock to plastic in an existing secondary trickling filter facility. On the other hand, if nitrification were required during December through March, treatment costs would increase signficantly since wastewater temperatures range from about 11°C to 14°C during this period in North Carolina.

The above discussions indicate that nitrification designs should take into account the wastewater temperatures prevailing during the periods when nitrification is required. Use of these generalized estimates, however, should be limited to screening and preliminary analyses. Actual design of AT facilities should be based on site-specified wastewater temperature data.

2.3 CBOD vs BOD

The presence of nitirifying bacteria in wastewater effluents can significantly affect the measured performance of treatment systems using the BOD_5 test procedure. Nitrifiers in the BOD test bottle can oxidize ammonia to nitrite and nitrate and thereby produce a greater measurement of the oxygen demand than what would occur without nitrifiers. A detailed discussion of this issue is presented in the proposed Secondary Treatment Information Regulation[8]. Additional information is also contained in "Standard Methods for the Examination of Water and Wastewater" fifteenth addition[9] and a paper entitled "Nitrification in BOD_5 Test Increases POTW Noncompliance"[10].

As discussed in these references, nitrifying bacteria can be found in wastewater from both secondary and advanced treatment systems. Therefore, the possible occurrence of significant amounts of nitrogenous oxygen demand (NOD) in the BOD measurements should be considered when evaluating the performance of existing and proposed treatment systems. Some AT reviews have shown that use of unadjusted BOD_5 performance data during warm months has led to the underestimation of the performance of existing facilities. Thus, some cases led to proposed AT projects with unnecessary nitrification facilities or capacity.

Suspended solids data can be used to detect the possible occurrence of NOD where only BOD_5 data is available. Effluent data for secondary treatment facilitites collected for the secondary treatment regulation indicates that $CBOD_5$ concentrations are generally between 0.6 and 0.85 of SS concentrations. Therefore, performance data showing BOD_5 concentrations significantly above SS concentrations suggests NOD might have influenced the BOD measurement. This relationship can be used to screen performance data to assess the possible existence of NOD in BOD measurements.

2.4 Costs and Pollutant Reduction Capabilities of Nitrification and Tertiary Filtration

The most commonly proposed processes for reducing $CBOD_5$ and ammonia concentrations beyond secondary treatment levels are various types of nitrification systems and/or tertiary filtration. About 90 percent of the proposed AT projects reviewed at EPA Headquarters included nitrification; about 60 percent included tertiary filtration. Nitrification following secondary treatment removes more UOD than tertiary filtration and effluent limitations sometimes require meeting both DO and ammonia criteria. For both of these reasons, secondary processes are usually followed by nitrification. Proposed tertiary filtration often followed nitrification and was seldom proposed as an add-on to a secondary treatment facility.

Funding was deferred for approximately 70 percent of the tertiary filtration processes proposed. Three principal reasons are: (1) nitrification alone was found sufficient to meet required $CBOD_5$ levels, (2) the limited

$CBOD_5$ reduction attainable would not significantly improve water quality or (3) the calculated water quality improvement was found to be within the margin of error of the WLA analysis.

Little data are currently available on the $CBOD_5$ removal capabilities of well operated and designed nitrification facilities. Table 2-4 is a preliminary summary of summer performance data for AT facilities collected by the AT Task Force. The Facility Requirements Division is currently involved in the preparation of a report that will summarize the performance and design characteristics of over 100 AT facilities currently in operation in the U.S. At least one year of influent and effluent data is being collected, and the performance of the facility will be summarized as a function of wastewater temperature and various design characteristics. Completion of a draft report is planned for February 1985.

The flow diagram of a typical AT facility featuring nitrification and tertiary filtration is shown in Figure 2-4. This analysis assumes that nitrification reduces 30 day average $CBOD_5$ concentrations in secondary treatment effluent from about 25 mg/l to about 10 mg/l; see Table 2-4. In general, during warm weather, nitrification will reduce ammonia concentrations by about 90 percent from about 20 mg/l to about 2.0 mg/l. Thus, nitrification would typically reduce UOD concentrations from about 130 mg/l ((25x1.5)+(20x4.57)) to 30 mg/l ((10x2.0)+(2.0x4.57)). Therefore, the total UOD reduction from nitrification would be about 100 mg/l.

Of the assumed 10 mg/l $CBOD_5$ remaining following nitrification, about 1.0 to 3.0 mg/l would be dissolved, and the remaining 7.0 to 9.0 mg/l of $CBOD_5$ would be suspended (EPA, 1982). Since tertiary filtration only removes suspended matter, tertiary filtration could remove about 7.0 to 9.0 mg/l of $CBOD_5$. Typically, tertiary filtration reduces non-soluble $CBOD_5$ from about 8.0 mg/l to about 3.0 mg/l. Therefore, total $CBOD_5$ removed by tertiary filtration would approximate 5.0 mg/l. Since tertiary filtration does not remove ammonia, the total UOD reduction provided by tertiary filtration would average about 10 mg/l (assuming a $CBOD_u$:$CBOD_5$ ratio of 2.0). If effluent $CBOD_5$ following nitrification were assumed to be 15 mg/l rather than 10 mg/l, including about 10 mg/l non-soluble $CBOD_5$, total $CBOD_5$ removed by tertiary filtration would be about 7 mg/l. The total UOD reduction with filtration is estimated to be about 14 mg/l, compared to 100 mg/l with nitrification. Therefore, nitrification removes between 8 and 10 times more UOD than tertiary filtration.

Figure 2-5 shows the marginal costs per mg/l of UOD removed by secondary, nitrification, and tertiary filtration systems (Engineering News Record Index 3900). In this example, the estimated present worth costs are $2.6 million for nitrification and $3.3 million for tertiary filtration. These estimates are based on data provided in EPA[3] for a new 10 MGD facility; adjusted upward by 30 percent to account for inflation. Nitrification costs represent a single-stage activated sludge nitrification system operated six months per year. Tertiary filtration costs represent a dual media tertiary filtration process operated six months per year.

As shown on Figure 2-5, secondary treatment for this facility would cost about $34,800 per mg/l of UOD removed. Since nitrification removes a large amount of NOD as well as additional CBOD with relatively low additional costs (i.e., costs for additional aeration tankage, blowers, etc.), the marginal cost for nitrification is only about $23,200 per mg/l of UOD removed. Tertiary filtration removes a relatively small amount of CBOD and no soluable NOD. Therefore, its marginal cost is relatively high at about $300,000 per mg/l of UOD removed.

Although these costs and UOD removal capabilities will vary for specific projects, the above analysis indicates that nitrification provides a much greater reduction in UOD at a lower marginal cost than tertiary filtration. Therefore, where filtration is proposed, as an add on to nitrification, the proposed filtration process should be supported by a thorough site-specific analyses of the prospective pollutant reductions and water quality enhancements.

2.5 Dissolved Oxygen Improvements Provided by Tertiary Filtration

Although tertiary filtration removes a relatively small amount of UOD at a relatively high marginal cost compared to ammonia removal, the primary basis for an EPA funding decision is whether water quality would be significantly improved by the filtration process.

The DO improvement from tertiary filtration is considered "significant" if this increase is definitely needed to achieve the designated uses or to substantially contribute toward attainment of the designated uses. Thus, if modeling uncertainties related to Ka and Kd rates prevent such a determination, additional data may be needed to support effluent filtration. Filtration would be justified if data are available to support estimates of Ka and Kd and thus minimize model uncertainty.

Although Ka measurements are often not needed to determine the need for nitrification, particularly on effluent dominated streams, Ka measurements may be needed to reduce model uncertainty and accurately define the DO improvement from tertiary filtration. Ka can be measured at the projected design flow using the ethylene or krypton methods or estimated using empirical formulae. However, insufficient data is presently available to define the Kd rate that should be used to predict the DO improvement from effluent filtration. Theoretically, the Kd rate used in this equation is the difference between the Kd with the filter and the Kd without the filter. Unfortunately, little intensive survey data has been collected below AT facilities with filtration to make this determination. Neither the bottle rate (Kl) of the effluent or the stream rate (Kd) determined during model calibration (at less than advanced treatment) should be used to estimate this Kd.

2.6 Tertiary Filtration to Improve Reliability

The improved reliability that tertiary filtration can provide to the treatment system was considered as part of the water quality justification for project submitted to the AT Task Force for review. The variability in effluent $CBOD_5$ is reduced by tertiary filtration through reduction of the maximum concentrations measured during a given period of time. This can result in an improvement in compliance with a given permit requirement when these daily concentrations are averaged for the period of time designated by the permit. However, no analyses have been conducted on tertiary filtration to show the water quality improvement that may result from reduction in the variability of $CBOD_5$ concentrations.

EPA funding recommendations for tertiary filtration to improve reliability would depend on analyses demonstrating that the reduced effluent variability resulting from this process would reduce the frequency of potential low DO occurrences in the receiving water, and that this frequency reduction is necessary to achieve the designated use. In addition, tertiary filtration should be shown as the most cost-effective means for achieving the desired reliability improvement.

2.7 Tertiary Filtration to Remove Suspended Solids (SS)

In most cases, SS limitations for proposed AT projects were consistent with, and about the same as, CBOD removal capabilities of the treatment facility. Water quality analyses were seldom presented solely to meet SS limitations. AT processes such as tertiary filtration to reduce SS concentrations may be recommended for funding if site-specific information shows that the additional solids removal is needed to protect the designated use. For example, filtration might be needed if an unfiltered effluent discharge located above a fish spawning area was shown to adversely affect this area.

2.8 Upgrading Trickling Filters to Achieve Nitrification

Section 4.2 presents the cost estimates for upgrading both activated sludge and trickling filter treatment systems to achieve nitrification. Most of the AT projects with existing activated sludge systems featured expansion of this system to achieve nitrification. However, most AT projects involving existing trickling filters omitted their use in achieving nitrification. This section discusses some cost effective modifications of trickling filters to achieve nitrification.

The treatment efficiency of trickling filters depends on the organic loading rate, the type of filter media, the wastewater temperature and the recirculation rate. The literature [6,11] reports the following organic loading rates and treatment efficiencies for wastewater temperatures in the range of 20-25°C:

Treatment Level	Media Type	Organic Loading (# BOD_5/1000 ft^3/D)
secondary	rock	20-30
	plastic	50-60
	modular plastic	
nitrification	random plastic	20-25
	modular plastic	25-30

The above table shows about the same organic loading rates for secondary treatment using rock media and nitrification using plastic media. Therefore, if design flow approximates existing flow, the most cost-effective means for upgrading existing rock trickling filters to achieve nitrification at wastewater temperatures above 20°C may involve only changing the filter media from rock to plastic. This approach may also be cost effective where existing flows and loadings are below existing filter design capacity and a the design loading will be less than the organic loading rate required to achieve nitrification using plastic media (20-25 # BOD_5/1000 ft^3/day).

Where future flow increases would cause loading rates to exceed levels required for nitrification, or where nitrification is required at wastewater temperatures significantly below 20°C, structural changes may be necessary as well as media changes to achieve nitrification. Such structural changes could involve increasing the height of the trickling filter, or adding new trickling filters. Since the loading rate required to achieve secondary treatment using rock media is about the same as that for nitrification using plastic media, the required volume increase of the trickling filter structure to achieve nitrification at higher flows would be proportional to the projected flow increase (design flow minus existing flow). For example, if an existing rock trickling filter at design capacity were 10 feet deep, and a flow increase of 50 percent were predicted, nitrification could be achieved by changing the rock media to plastic, and increasing the height of the trickling filter to 15 feet. This would maintain a loading rate of 20-25 #BOD_5/1000ft^3/day.

Since the activity of nitrifying bacteria decreases at lower temperatures, structural modifications may be necessary where plastic media is used for nitrification at wastewater temperatures below 20°C. The organic loading rate needed to achieve a given treatment efficiency changes proportionally with changes in the wastewater temperature. For example, if an existing rock trickling filter currently at design capacity were upgraded to provide nitrification at 15°C, the loading rate to achieve nitrification would be about 30 percent less than that at 20°C. Therefore, the volume of the trickling filter would be increased by about 30 percent to provide for the required loading rate.

The decision on whether to use a trickling filter system for nitrification, or to use some other process such as activated sludge, depends on the specific characteristics of the existing trickling filter facility and the design wastewater flow and temperature. If the projected increase in wastewater flow is large or if nitrification must occur at low temperatures, then the cost for structural changes needed to provide adequate trickling filter capacity may exceed the cost of other alternatives such as a second-stage activated sludge system. Thus, in these cases it may not be cost-effective to provide nitrification using trickling filters. On the other hand, where the projected wastewater flow increase is low or moderate and only warm weather nitrification is needed, modifying the existing trickling filters as described above may be cost effective. These options should be checked where trickling filters exist and ammonia removal is required.

2.9 Use of Flocculation Channels for Reducing Effluent $CBOD_5$ and SS from Trickling Filters

Low efficiency of final clarifiers often adversely affects performance of trickling filter systems[12]. Poor settling in such clarifers often exists because the floc particles are extremely small (i.e., pin-point floc). Since about 50 percent of the $CBOD_5$ remaining in the trickling filter effluent prior to clarification is associated with suspended solids, the performance of trickling filter systems can be improved by enhancing the settling efficiency of the final clarifier.

Common approaches for improving solids removal of trickling filter systems are increasing clarification capacity, addition of chemical flocculants, or addition of tertiary filtration. Capital and operating costs of these approaches are substantial. A less costly approach suggested by the AT Task Force for one project, involves addition of a flocculation channel between the trickling filter and the final clarifier; (see Figure 2-6). Flocculation channels should be considered where the $CBOD_5$ removal performance of trickling filters must be improved.

The flocculation channel mixes the trickling filter effluent with sludge recycled from the secondary clarifier. Slow mixing in the channel causes agglomeration of the finely divided particles in the trickling filter effluent with the more settleable recycled sludge particles to achieve more efficient solids removal.

Figure 2-7 shows the improvement in effluent quality resulting from use of the flocculation channel for a Corvalis, Oregon treatment facility (9.7 MGD design flow)[13]. As shown, the conventional trickling filter system experienced frequent excursions from a secondary treatment level of 30 mg/l BOD_5. To achieve proposed BOD_5 permit limitations of 10 mg/l, a second-stage activated sludge unit and a flocculation channel were added. From March through December 1978, the second stage activated sludge system was placed on-line and BOD_5 concentrations were consistently reduced to less than 10 mg/l (referred to as "coupled mode" on Figure 2-7). Then, from January through June 1979, the activated sludge process was shut down and the trickling filter effluent was mixed with the secondary clarifier effluent in a flocculation channel with a retention time of about 10 minutes (shaded portions of Figure 2-7). As a result of the improved settling characteristics resulting from the flocculation channels, the BOD_5 concentrations remained below 10 mg/l without the second stage activated sludge unit.

2.10 Oxidation Ditches

AT reviews have shown oxidation ditches to be a cost-effective biological treatment process for achieving advanced levels of $CBOD_5$ and ammonia removal in newly installed municipal facilities ranging in size from 1.0 MGD to 10.0 MGD. Oxidation ditches to achieve nitrification were approved for the two AT projects featuring this process. Because of increased construction costs for larger facilities coupled with land costs for large site areas, conventionally designed oxidation ditches may not be cost-effective for larger installations. Although deep channel (greater than 10 feet deep) oxidation ditches may have lower capital costs than competing biological systems for facilities much larger than 10.0 MGD, the high operating costs to assume adequate aeration within the larger deep channels (i.e., greater than 10-15 MGD) may offset the lower capital cost.

Studies of secondary treatment level oxidation ditches indicate that $CBOD_5$ levels of approximately 12 mg/l can be achieved on a 30-day average basis during warm-weather conditions with a reliability of 80 percent without tertiary filtration; see Figure 2-8. Oxidation ditch facilities are also capable of nitrifying even at wastewater temperatures of 10°C, and can achieve a certain degree of denitrification. Nitrification and denitrification capabilities result from the long hydraulic retention times which allow solids retention times (SRT) ranging from 10 to more than 50 days.

The cost advantages of oxidation ditches result from omission of primary clarifiers and separate digestion facilities and from simplicity of construction. Even where a primary treatment facility exists, the oxidation ditch often provides a cost-effective upgrade to achieve secondary or advanced treatment levels (e.g. ammonia removal) for flows less than 10.0 MGD. Where secondary treatment facilities other than oxidation ditches exist, an oxidation ditch is seldom a cost-effective means for achieving advanced treatment levels.

2.11 Post-Aeration

In some AT projects reviewed by the Task Force, the DO improvement afforded by post-aeration was not evaluated. Low effluent DO concentrations can cause violations of DO criteria at the mixing point in the stream and can significantly impact the DO at the critical DO sag point downstream. Therefore, the water quality improvement that may result from post-aeration is evaluated for all AT projects.

Since stream and wastewater temperatures may be different, measurements of the stream temperature following mixing with the effluent is needed to determine the DO saturation below the outfall. The initial DO concentration in the receiving stream below the outfall can then be calculated by a mass balance of the effluent and stream temperatures.

The most significant factor affecting initial DO concentration in effluent dominated streams is the effluent DO concentration (i.e., assuming there are no significant upstream sources of UOD). If post-aeration is not provided, effluent DO from a typical secondary or nitrification plant could be about 0.5 mg/l. If post-aeration were provided, effluent DO would be about 6.0 to 7.0 mg/l. Thus, assuming a 1:1 dilution ratio and a background DO concentration of 8.0 mg/l, post-aeration would provide a DO improvement of 3.25 mg/l (from 4.25 to 7.5 mg/l) at the outfall.

The resulting deficit at the critical DO sag point associated with this initial deficit may be calculated using the following equation:

$$D = Do \exp(-Kat)$$

where,

D = DO deficit at the critical sag point caused by the initial DO deficit
Do = initial DO deficit
Ka = reaeration rate, 1/day
t = travel time to the critical sag point

Calculations using this equation show that in some cases the initial DO deficit comprises a significant part of the DO deficit at the critical point downstream. For example, assuming a reaeration rate of 3.0 per day and a travel time of 0.5 days, the 3.25 mg/l DO improvement at the outfall calculated above would result in a 1.0 mg/l improvement at the critical sag point. If the DO saturation is 8.0 mg/l, this 1.0 mg/l DO improvement represents 42 percent of the available deficit. Therefore, post-aeration may significantly improve DO or even prevent violations of DO criteria, particularly in effluent dominated streams. Further, post-aeration may be sufficient to meet DO criteria withour additional removal of CBOD or ammonia.

In general, post-aeration is a relatively inexpensive process for improving critical DO concentrations compared to other AT processes that remove CBOD and ammonia. Post-aeration via cascade aeration sometimes can achieve an effluent DO of 6.0 mg/l at wastewater temperatures of about 25°C. An effluent DO concentration of 6.0 mg/l is usually assumed for modeling. Diffused aeration would generally be required to consistently achieve higher effluent DO concentrations at this temperature. Where biological treatment systems use diffused aeration for activated sludge treatment, it may be relatively inexpensive to post-aerate using some of this aeration capacity to achieve effluent DO concentrations of about 7.0 mg/l. On the other hand, without such aeration capacity, providing post-aeration facilities to achieve consistent effluent DO concentrations of greater than 6.0 mg/l at 25°C may not be cost effective.

Table 2-1

ACTIVATED SLUDGE PLANT
UPGRADING TO NITRIFICATION

Capital Cost Summary $\times 10^5$

Temp	Case	Flows MGD 1	5	10	20	50
5	1	1.4	5.4	10.0	18.0	-
5	2	1.1	3.8	6.7	11.5	25.8
5	3	1.1	3.5	6.2	10.6	22.1
10	1	0.8	2.8	5.2	9.5	21.0
10	2	0.7	2.1	3.7	6.4	13.3
10	3	1.0	2.8	4.7	7.8	16.3
15	1	0.4	1.3	2.3	4.1	9.5
15	2	0.5	1.3	1.9	3.1	5.8
15	3	0.8	2.3	3.7	6.2	12.3
20	1	0.14	0.4	0.7	1.2	2.7
20	2	-	-	-	-	-
20	3	0.73	2.0	3.3	5.1	10.3

Operation and Maintenance Costs Summary $\$ \times 10^6$

Temp	Case	1	5	10	20	50
5	1	.08	.34	.65	1.24	-
5	2	.06	.23	.42	.78	1.85
5	3	.05	.17	.32	.59	1.35
10	1	.04	.17	.32	.62	1.46
10	2	.04	.11	.23	.42	.94
10	3	.04	.14	.25	.45	1.01
15	1	.03	.07	.13	.25	.61
15	2	.03	.08	.14	.20	.40
15	3	.04	.12	.20	.36	.78
20	1	.02	.04	.06	.10	.14
20	2					
20	3	.04	.11	.18	.30	.64

Table 2-2

TRICKLING FILTER PLANT
UPGRADING TO NITRIFICATION

Capital Costs Summary $ x 10^6

				Flow MGD		
Temp	Case	1	5	10	20	50
>22	1	.2	1.3	2.5	5.0	12.5
>15	2	.3	1.4	2.7	5.4	13.5
>15	3	.8	3.5	7.2	14.3	35.0
5	4	1.2	4.5	8.4	15.2	30.8
10	4	0.9	3.2	6.0	11.0	25.7
15	4	.8	2.5	4.8	8.5	19.3
5	5	1.1	3.5	6.2	10.6	22.1
10	5	1.0	2.8	4.7	7.8	16.3
15	5	0.8	2.3	3.7	6.2	12.3
20	3	0.73	2.0	3.3	5.1	10.3

Operation and Maintenance Costs Summary Annual - $ x 10^6

				Flow MGD		
Temp	Case	1	5	10	20	50
>22	1	0	0	0	0	0
>15	2	.05	.18	.36	.74	1.7
>15	3	.03	.11	.22	.42	1.1
5	4	.03	.11	.20	.38	.88
10	4	.03	.10	.19	.36	.84
15	4	.02	.10	.18	.35	.82
5	5	.05	.17	.32	.59	1.35
10	5	.04	.14	.25	.45	1.01
15	5	.04	.12	.20	.36	.78
20	3	.04	.11	.18	.30	.64

Table 2-3

Wastewater Temperature Data in Selected States

Temperature Statistics

Mean is Avg. Monthly Mean Temperature (Degrees C)
# of obs. is number of monthly observations

State	Stat.	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Florida	Mean	18.4	17.8	20.4	21.8	22.5	23.5	23.5	23.5	23.4	22.1	21.7	18.7
	# of obs.	35	34	33	33	37	36	35	46	42	41	41	42
Indiana	Mean	9.8	8.8	13.6	15.9	19.6	21.9	22.7	22.5	20.5	17.2	14.1	11.7
	# of obs.	14	13	14	13	14	13	14	14	13	14	13	12
Iowa	Mean	8.5	8.1	12.5	16.3	19.7	22.3	22.7	22.6	20.8	17.1	13.4	16.9
	# of obs.	26	25	26	25	27	28	29	29	28	27	28	27
Kansas	Mean	11.3	10.6	14.0	17.6	20.6	23.9	23.5	23.4	22.2	18.4	14.6	12.8
	# of obs.	19	19	20	20	20	20	21	16	27	32	40	38
Michigan	Mean	8.5	7.9	11.2	14.6	17.7	19.7	21.6	21.1	19.3	15.4	12.5	10.2
	# of obs.	63	61	64	62	61	64	64	62	64	62	64	61
Minnesota	Mean	-	-	-	-	-	-	-	-	-	-	-	-
	# of obs.	0	0	0	0	0	6	0	0	4	0	0	3
Missouri	Mean	-	-	-	-	-	-	-	-	-	-	-	-
	# of obs.	2	2	2	2	2	2	2	2	2	2	2	2
New York	Mean	8.0	7.9	8.0	10.3	14.4	17.1	19.9	21.3	19.9	16.6	13.1	10.4
	# of obs.	71	71	73	72	76	75	74	76	76	75	75	73
N. Car.	Mean	13.3	12.1	14.7	17.6	20.7	22.7	25.2	26.2	24.8	21.3	18.2	15.0
	# of obs.	18	18	18	17	17	16	17	17	17	17	17	17
Ohio	Mean	-	-	-	-	-	-	-	-	-	-	-	-
	# of obs.	0	0	0	0	4	0	0	0	0	0	0	1
Okalahoma	Mean	13.6	11.5	13.9	16.5	19.4	23.7	27.1	27.4	26.2	22.0	19.3	15.8
	# of obs.	6	6	5	6	6	5	6	6	5	6	6	5
Tennessee	Mean	12.7	12.5	15.0	17.1	18.9	21.1	22.4	22.5	21.9	18.9	16.5	14.7
	# of obs.	28	27	25	27	27	27	24	26	26	28	27	25
Texas	Mean	15.7	15.7	17.8	19.9	22.3	24.8	26.2	26.4	25.7	22.1	19.1	16.9
	# of obs.	26	21	21	21	21	21	21	21	21	20	20	21
Virginia	Mean	11.1	10.9	13.7	16.8	18.3	20.0	22.2	22.8	21.5	18.0	15.5	13.2
	# of obs.	36	36	38	39	41	41	42	42	55	51	50	50
Washington	Mean	9.5	12.4	14.3	16.0	17.6	19.2	-	28.7	-	-	13.4	-
	# of obs.	16	16	15	15	14	16	14	15	15	15	16	14
Wisconsin	Mean	7.4	7.5	10.7	14.8	18.3	20.1	22.3	21.2	19.6	15.3	12.4	9.5
	# of obs.	48	50	50	49	49	50	49	51	49	45	50	52

Table 2-4

Nitrification Facilities[1]
Comparison of Summertime CBOD$_5$ and BOD$_5$ Effluent Data

Name	Flow-MGD Exst	Flow-MGD Design	Process	BOD$_5$ (mg/l)	CBOD$_5$ (mg/l)	TSS (mg/l)	%NOD	NH$_3$ (mg/l)
Chicago (NS) IL	333	333	AS	6	3	7	50%	0.7
Chicago (WSW) IL	900	900	AS	7	4	7	43%	0.6
E Fitchberg MA	7	12.4	AS	16	7	20	57%	2
Ft. Collins CO	7	6	AS	12.6	2.5	-	80%	2.5
Franklin OH	3.4	4.5	AS	9.4	6.6	9	35%	1.8
Grand Is. NB	6	7	AS	20	5	-	75%	4
Grand Ledge MI	-	-	AS-Phos	12	2.5	-	78%	0.6
Jackson MI	14	19	AS-Phos	9	4	7	56%	2
Lawrence KS	6	9	AS	22	6.4	7	71%	5(W)
Malborough MA	-	5.5	AS-Phos	-	3.4	6.4	-	0.5
Michigan City IN	12	15	AS-Phos	7	3	14	60%	3
Middletown OH	18	23	AS	6.4	3.7	6	45%	1.0
Pittsfield MA	10	17	TF-AS	-	3	6	-	0.5
Roanoke VA	21	35	AS	3.2	-	8	-	0.1
Washington DC	309	309	AS	12	4	10	67%	0.5
W.Contra Costa CA	10	12.5	TF-AS	35	5	6	86%	11(W)

(1) More that 75% NH$_3$ removal

(W) Winter average

AS-Activated Sludge

TF- Trickling Filter

Phos-Chemical addition TP removal

FIGURE 2-1 COSTS OF ALTERNATIVE NITRIFICATION SYSTEMS USING ACTIVATED SLUDGE FOR DIFFERENT WASTEWATER TEMPERATURES

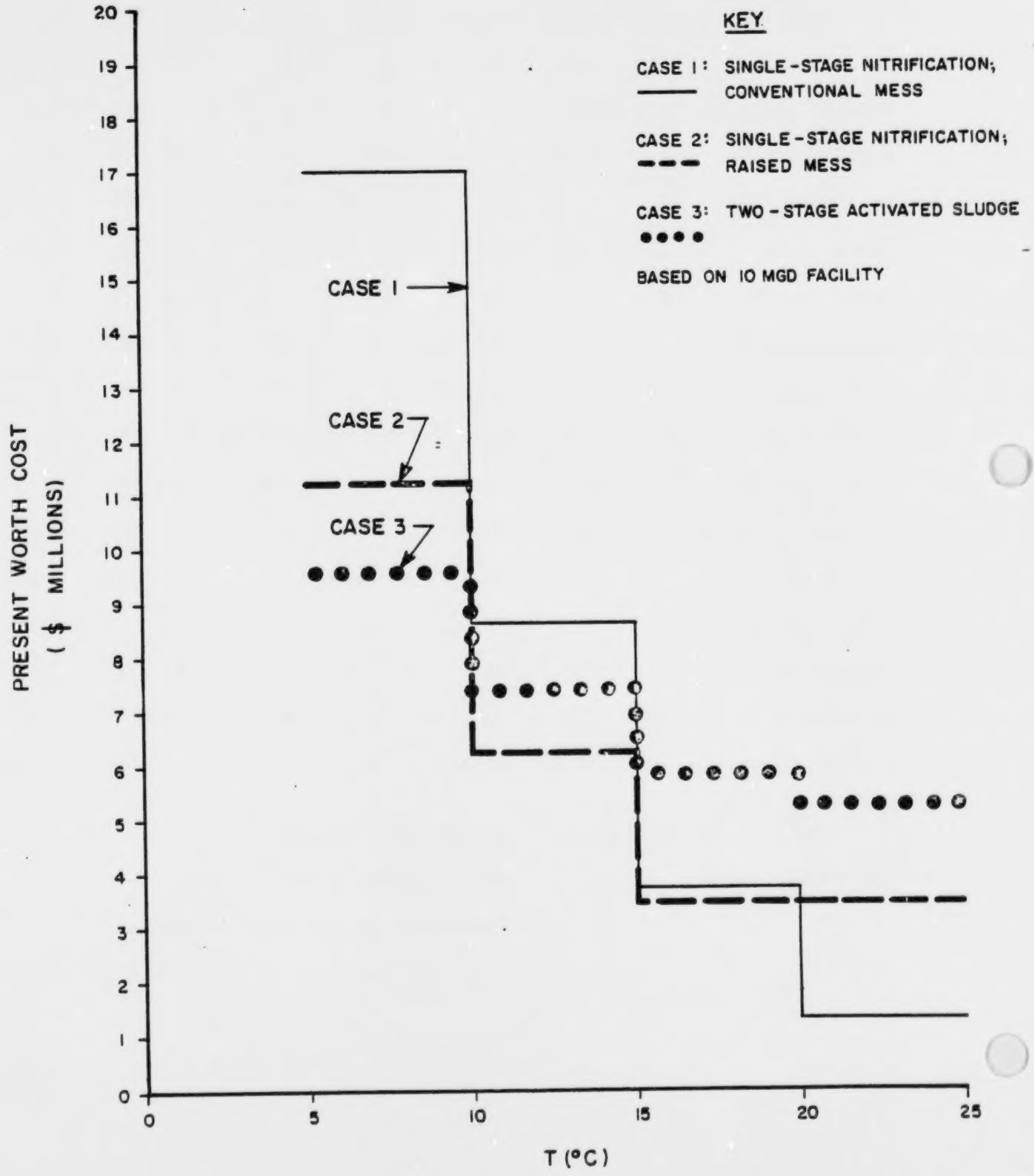
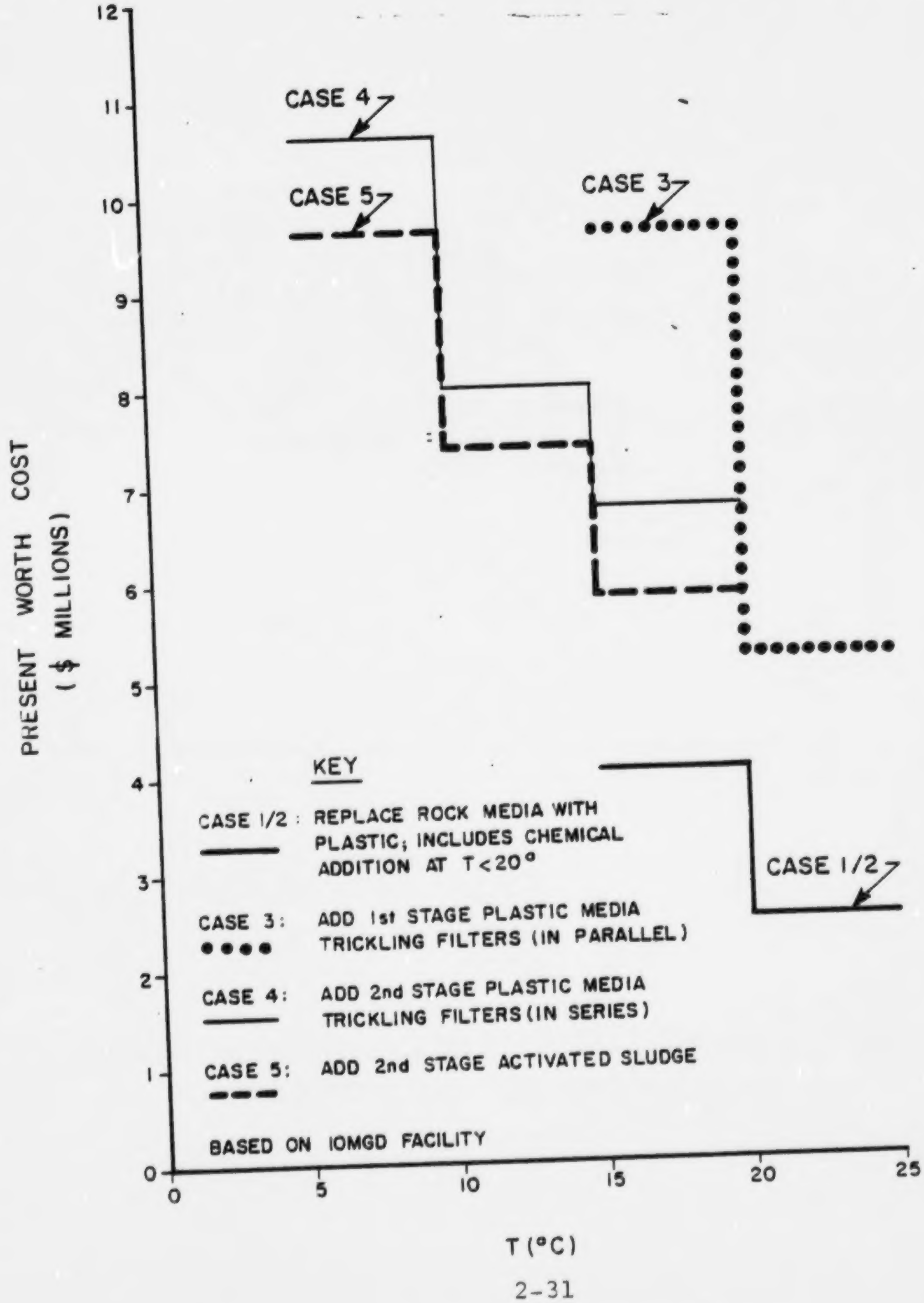


FIGURE 2-2. COSTS OF ALTERNATIVE NITRIFICATION SYSTEMS USING TRICKLING FILTERS FOR DIFFERENT WASTEWATER TEMPERATURES



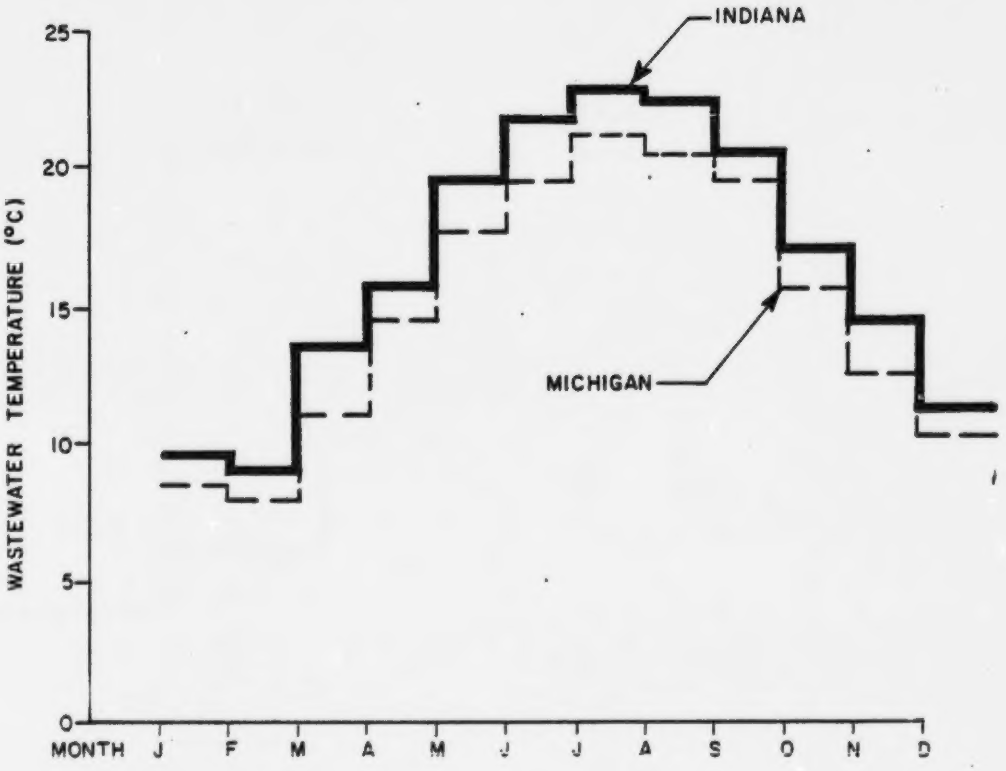
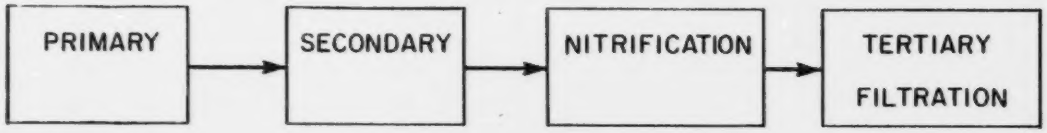


FIGURE 2-3 MONTHLY CHANGES IN WASTEWATER TEMPERATURE IN INDIANA AND MICHIGAN

FIGURE 2-4

Single Stage Nitrification — Typical Removal Efficiencies and Present Worth Costs


PRIMARY
SECONDARY
NITRIFICATION
TERTIARY FILTRATION

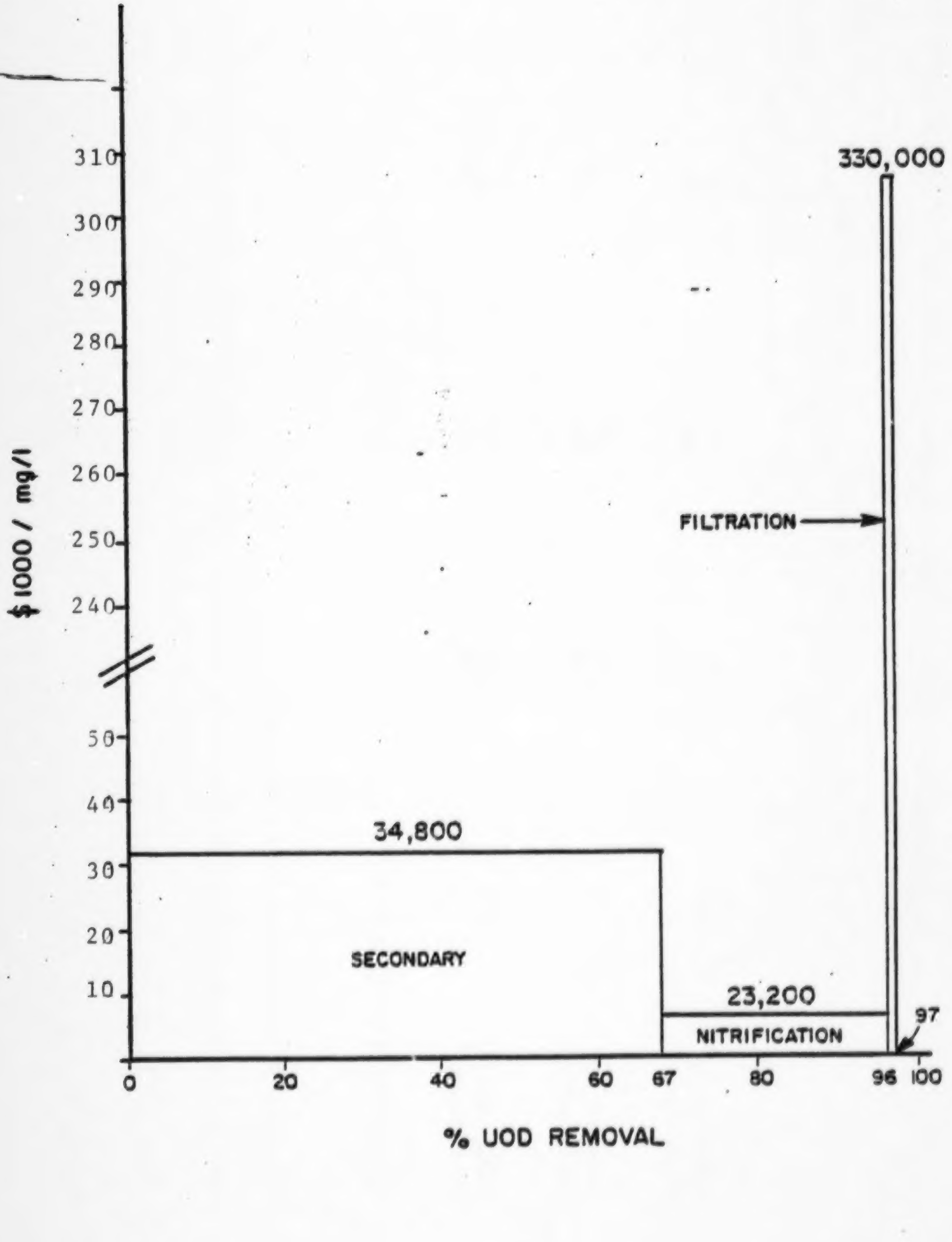
$CBOD_5$ (mg/l) (% Removal)	200	130 35%	25 88%	10 95%	5 98%
NH_3 (mg/l) (% Removal)	25	25 0%	20 20%	2 92%	2 92%
UOD (mg/l)* (% Removal)	414	309 25%	141 65%	29 93%	19 95%
Incremental Cost ($ million)**			9.5	2.6	3.3

*UOD = 2.0 ($CBOD_5$) + 4.57 (NH_3-N)

**Based on 10 mgd facility, present worth; assumes 6 month operation of AT processes

FIGURE 2-5

MARGINAL COSTS FOR SECONDARY, NITRIFICATION AND TERTIARY FILTRATION — TYPICAL 10 MGD SINGLE-STAGE ACTIVATED SLUDGE FACILITY



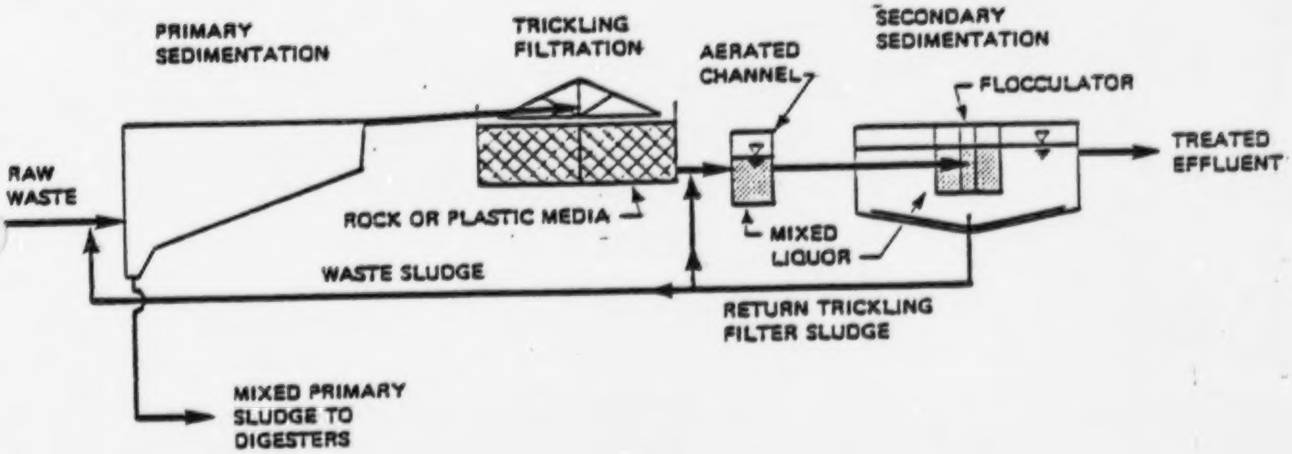

PRIMARY SEDIMENTATION
TRICKLING FILTRATION
AERATED CHANNEL
SECONDARY SEDIMENTATION
FLOCCULATOR
RAW WASTE
TREATED EFFLUENT
ROCK OR PLASTIC MEDIA
MIXED LIQUOR
WASTE SLUDGE
RETURN TRICKLING FILTER SLUDGE
MIXED PRIMARY SLUDGE TO DIGESTERS

Figure 2-6 Trickling Filter Solids Contact Process

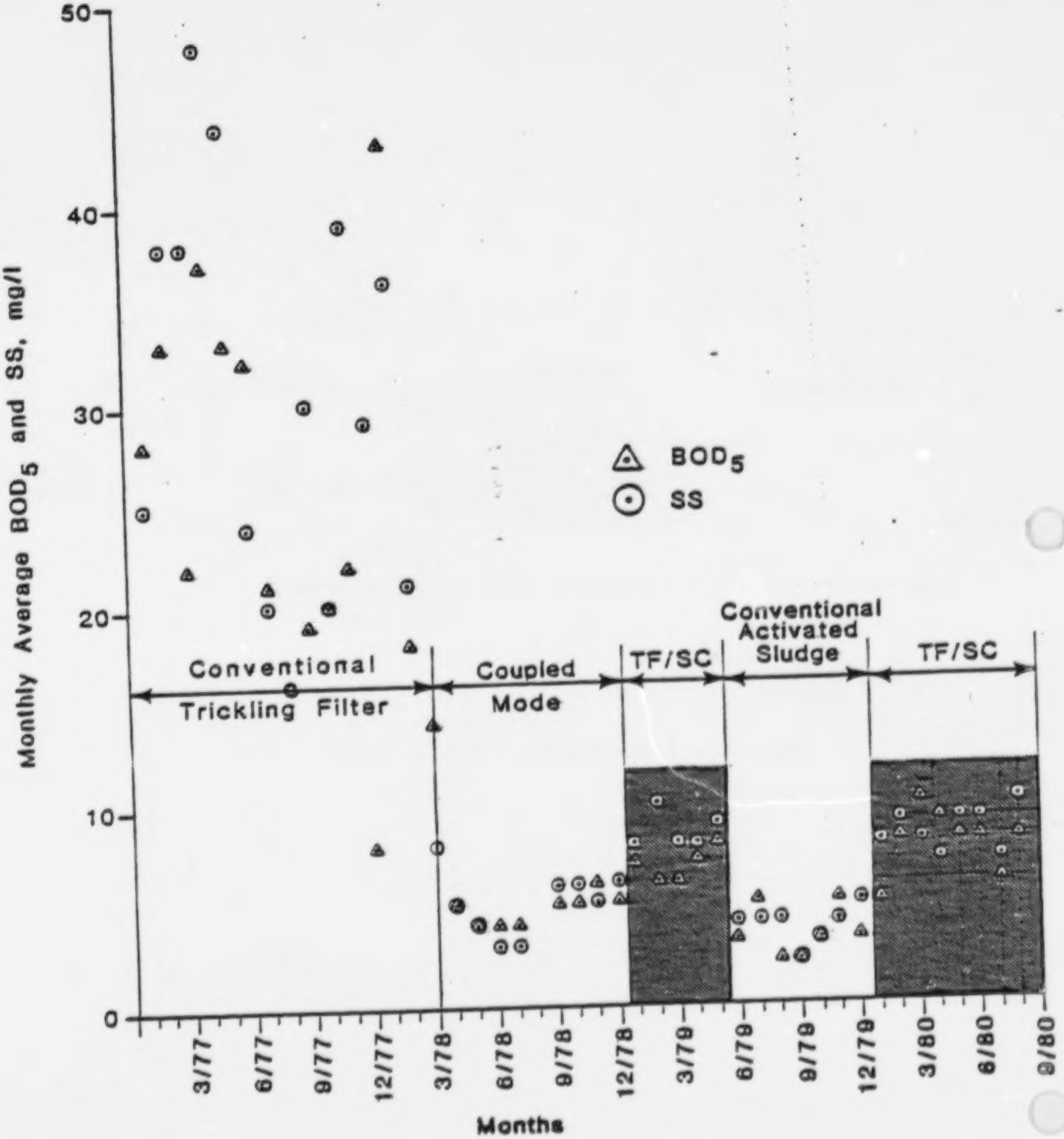


Figure 2-7 Effluent BOD_5 and Suspended Solids Concentrations, January 1977 through August 1980

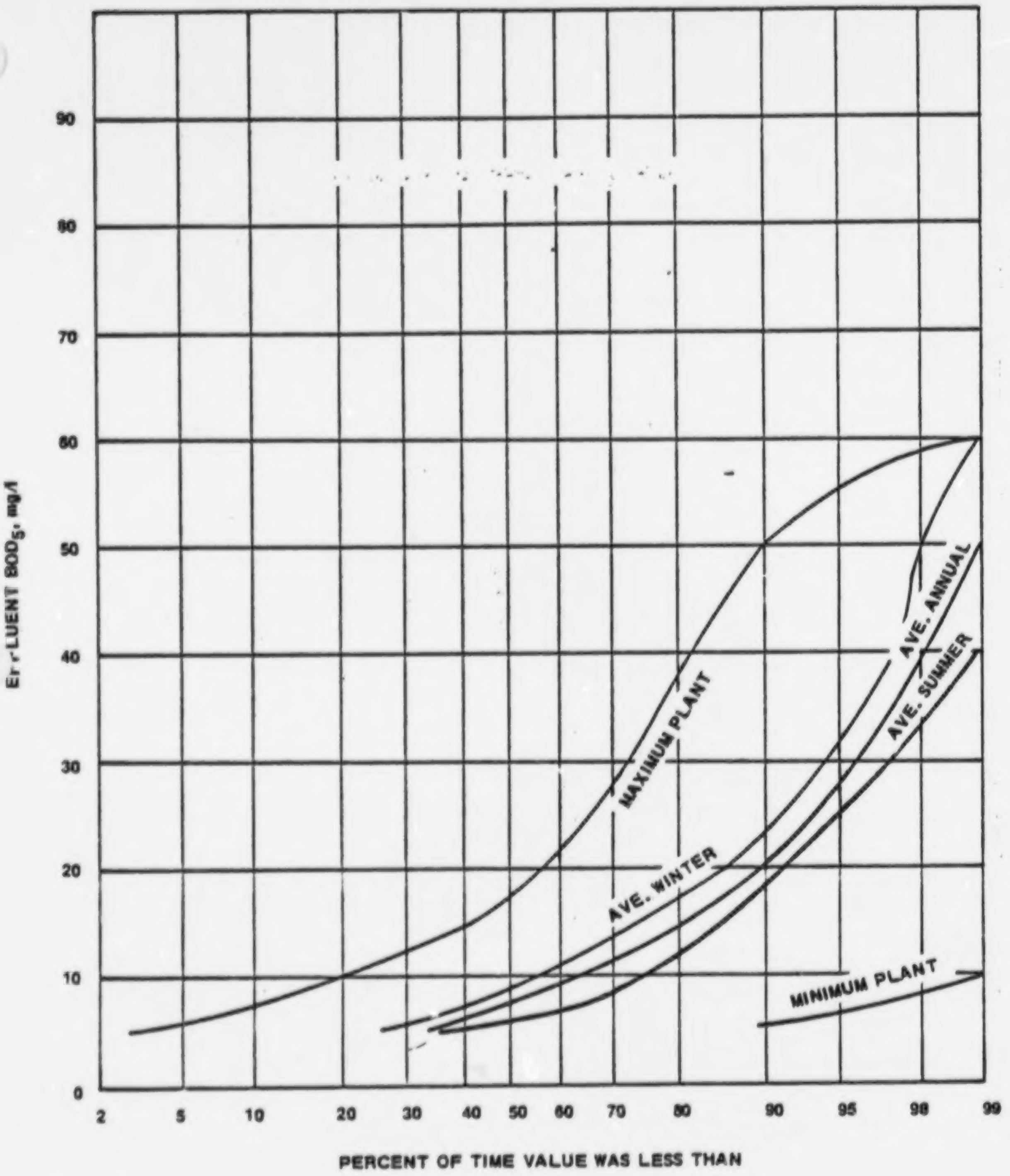


Figure 2-8 Oxidation ditch plant BOD$_5$ reliability.

3.0 Water Quality Issues - Dissolved Oxygen (DO)

This section describes DO water quality issues encountered in AT reviews. The information herein is intended to supplement but not unnecessarily duplicate available literature and guidance on water quality standards, wasteload allocations and permits. This section should be used together with the Technical Guidance Manuals[14], the Simplified Mathematical Modeling Methodology[15] and Addendum[16] and the Water Quality Standard Handbook[17] available from the EPA, Office of Water Regulations and Standards, Washington, D.C. The basic theoretical framework and modeling equations contained in these and other technical references is not repeated in this section. As an introduction to the discussion of specific DO issues, the general approach usually followed by the Headquarters AT Task Force for the water quality portion part of the AT review is briefly outlined below.

The first step in the review of the water quality analyses supporting the need for advanced treatment usually involves the review of water quality standards. Beneficial uses are generally not questioned in an AT review. However, the review considers factors such as physical constraints of the waterbody or uncontrollable nonpoint source pollution to ensure such factors would not prevent attainment of the water quality improvements predicted to result from AT. DO and other criteria are checked to determine whether they are more stringent than EPA criteria and were developed according to EPA guidelines for deriving site-specific water quality criteria[17]. EPA or site-specific criteria provide the basis for the EPA funding decision on the proposed AT processes.

The second step features the review of the analytical framework for predicting the DO and other water quality improvements that will result from each AT process. A mathematical model employing the basic Streeter/Phelps equations is commonly used to determine the level of treatment needed to meet DO criteria. If a technically complete DO modeling analysis shows that a proposed AT process is definitely needed to achieve significant water quality improvement, AT funding is recommended. This funding recommendation will reflect consideration of the technical uncertainties of the analysis, the degree of improvement predicted and the estimated cost of each AT process.

Finally, AT effluent limitations are reviewed to ensure the AT limits are appropriate; they are established taking into account seasonal changes in flow and temperature and they correspond with appropriate permit averaging periods (generally 30 days).

3.1 Dissolved Oxygen Criteria

Dissolved oxygen (DO) criteria proposed for most AT projects were found to be consistent with EPA DO criteria (1976)[18] and recent draft revisions. Thus, in most cases, protection of warmwater fisheries was based on a 5.0 mg/l daily average DO criterion and a 6.0 mg/l daily average criterion was applied to protect coldwater fisheries. As part of AT reviews, criteria are checked to ensure that they are consistent with the designated uses.

3.2 Reaction Rates

3.2.1 Carbonaceous Deoxygenation Rate (Kd)

Kd values used in modeling for AT projects varied widely because Kd depends largely on the unique characteristics of the wastewater and the receiving water and their combined effect on the rate of CBOD decay. In some cases Kd values were selected without site-specific supporting data. While site-specific data or data from similar streams are not always essential, such data are needed where expensive AT processes are proposed or where unusual wastewater or receiving water characteristics are involved.

The major issues encountered in AT reviews affecting the estimation of Kd are: (1) the variability of Kd as a function of the stability of the municipal and industrial source CBOD and (2) algal effects on Kd.

Although Kd is primarily affected by physical factors of the stream (e.g. depth and bottom characteristics), biological stability of the wastewater can also affect Kd. For some proposed AT projects, the Kd rate set without receiving water data was consistent with that expected for a typical municipal effluent with no stream data, but was applied to an effluent influenced by industrial wastewater. Kd values influenced by industial wastewater with more stable CBOD (compared to municipal wastewater) are likely lower than those influenced by municipal domestic wastewater. Similarly, the expected Kd would be higher than that for municipal wastewater if the industrial component is less stable and therefore readily oxidized. In general, where the industrial wastewater component is significant site-specific data is needed to establish Kd.

Projection model runs for different levels of treatment should also account for CBOD stability. For the AT projects reviewed, the Kd rate was seldom adjusted to reflect wastewater differences between calibration and projection model runs and for the different levels of treatment. Since Kd values for a given river system can only be measured for the existing treatment level, Kd selected for simulating the effects of higher treatment levels cannot be determined from actual site-specific data. Unfortunately, Kd values have seldom been measured after the installation of AT processes. This lack of data complicates the estimation of Kd values associated with different levels of AT.

A recent EPA Office of Water Program Operations study, "Before and After Comparisons of Water Quality following Municipal Treatment Plant Improvements"[19], evaluates the change in Kd following installation of higher treatment levels. Although adequate data were available for only a few treatment facilities, analyses of available data suggested that Kd associated with nitrified effluents averages about 60 percent of that for a secondary discharge. While this limited data do not conclusively define this relationship, the data indicate that Kd values associated with AT levels are generally lower than Kd values determined for lesser treatment levels. Thus, sensitivity analyses for AT model runs should encompass a Kd range that includes a Kd that is substantially lower than the Kd rate determined from model calibration.

Algae can affect the CBOD data used to calculate Kd. For some AT projects, chlorophyll "a" concentrations in free flowing streams exceeded 100 ug/l and reached as high as 800 ug/l for one project. Algal respiration and decay in the CBOD bottle can cause higher measured CBOD values and thus higher Kl rates compared to samples without algae. In addition, if the concentration of suspended algae (i.e., algae that would get into the BOD bottle) is not constant in the stream below the discharge, the measured CBOD concentrations would not indicate a defined rate of decay (Kd). Where the concentration of suspended algae increases in the stream, there may be a net increase of measured CBOD below the discharge.

Since the presence of algae complicates the estimation both Kl and Kd rate, a laboratory procedure to minimize these effects is necessary. Such a procedure involves measuring filtered CBOD together with total CBOD at several locations downstream of the discharge. The oxygen demand resulting from settleable (i.e., filtered) organics is then accounted for separately in establishing the Kr (settling) and SOD rates. However, even with filtered and unfiltered CBOD data, it is difficult to select model coefficients for calibration and projection modeling for algae dominated streams. Where large concentrations of algae occur in the receiving stream, a range of Kd rates should be estimated based on filtered and unfiltered CBOD data. Some general rules of thumb follow:

- Algal impacts on Kd occur wherever high concentrations of chlorophyll "a" or large diurnal DO fluctuations occur.
- 10 ug/l of chlorophyll "a" will increase the $CBOD_u$ concentration by 1 mg/l above that without algae. Rough estimates may be obtained from multiples of this relationship; i.e., 100 ug/l of chlorophyll "a" may increase the $CBOD_u$ concentration by 10 mg/l.
- If the stream is effluent dominated with most of the CBOD from the discharge rather than algae, do not filter. If the stream is not effluent dominated and most of the CBOD is from algae, run filtered and unfiltered samples.

3.2.2 Nitrogenous Deoxygenation Rate (Kn)

Biological nitrification for ammonia reduction is the AT process most often proposed in the AT projects reviewed. With the exception of effluent dominated streams with ammonia removal requirements at temperatures above 20°C (see the AT policy[1]), the AT Task Force carefully evaluates the Kn rate used in modeling. Several factors related to ammonia makes Kn selection one of the most important aspects of DO analysis and model development. These factors are:

- Approximately 75 percent of the total UOD removed by the nitrification process is associated with ammonia; 25 percent is associated with CBOD.
- The Kn rate is the only mechanism used to simulate the nitrogenous oxygen demand in most models.
- Nitrifying bacteria are extremely sensitive to environmental conditions such as temperature, benthic conditions and hydrologic conditions. Therefore, their activity will vary for different times of the year and for different waterbodies.
- The cost of ammonia removal is much higher during cold weather than during warm weather.

The major Kn related issues encountered in AT reviews are: (1) the variability of Kn as a function of physical characteristics of the waterbody, (2) algal effects on Kn and (3) seasonal analyses of Kn and ammonia removal requirements.

For some AT projects reviewed, Kn rates applied to deep and slow moving rivers, without site specific data, were higher than those normally expected for such rivers. The availability of surfaces for nitrifier attachment can affect Kn rates. These surfaces include the stream bottom and suspended particles in the water column. Therefore, shallow streams with rocky bottoms favor the growth of nitrifying bacteria with associated high Kn rates. Deep rivers composed of sands, silts or clays generally have fewer attached nitrifiers. Although these streams may support significant populations of nitrifiers in the water column, they tend to have lower Kn values than shallow streams. Since deeper streams usually have lower Kn rates and provide more dilution than shallower streams, the need for ammonia removal is less definitive. Therefore, large and deep streams require site-specific data and a rigorous determination of the Kn rate in the modeling analysis to support ammonia removal requirements.

Kn rates determined for shallow, fast moving streams were highly variable for the many AT projects that proposed ammonia removal to meet DO criteria. Kn rates ranged from 0.1 to 1.5 per day for proposed AT projects that discharge to such streams. Since small and shallow streams tend to be effluent dominated, the DO improvement from warm weather nitrification is usually significant even with relatively low Kn rates. However, since the Kn rates on such streams is highly variable, the selection of an appropriate Kn rate should be based on site-specific nitrogen series data.

Large concentrations of algae, either suspended or attached, markedly affect Kn. For most AT projects reviewed, the selected Kn reflects only the loss of ammonia. Such an approach results in the overestimation of Kn where significant algal effects occur. Algae consume ammonia as a nutrient. Therefore a Kn rate calculation based only on the loss of ammonia would include uptake of ammonia by algae as well as ammonia oxidation. In some cases, use of the rate of increase of nitrate is a better approach for estimating Kn because nitrate increase results directly from oxidation of ammonia in the stream. However, as a cautionary note, under some conditions, algae can uptake nitrate as well as ammonia. Therefore, the Kn rate derived from nitrate data would represent the minimum Kn. If the model shows that ammonia removal is needed to meet standards based on the minimum Kn derived from nitrate data, then a nitrification process would be justified. Therefore, selection of a Kn rate for modeling should stem from a sensitivity analysis using a range of Kn values reflecting changes in all forms of nitrogen. The rate of nitrate production would represent the minimum Kn.

An example of the effect of algae in determining Kn values is provided in Figure 3-1. This Figure is a plot of ammonia and nitrate concentrations at the site of a proposed nitrification facility. The Figure shows a distinct reduction in ammonia concentrations, particularly between river miles 12 and 8 and again from miles 8 through 6. Based on this decrease in ammonia, a Kn value of 1.0 per day can be calculated. Although no chlorophyll 'a' data were available, DO data show diurnal fluctuations of 4-6 mg/l. This indicates the presence of algae or periphyton capable of uptaking a significant amount of ammonia. Thus, the apparent 1.0 per day Kn rate likely reflects ammonia uptake by algae and aquatic plants, as well as ammonia oxidation.

Figure 3-1 also shows nearly constant nitrate concentrations throughout the stream reach. As a result, the 1.0 per day Kn rate predicted above exceeds by 5 times that derived from the actual nitrate data. Accordingly, uptake of ammonia by algae instead of ammonia oxidation may have caused the reduction in ammonia concentration.

A critical issue in determining the Kn rate involves the time of the year these rates are determined and applied. Although ammonia and nitrate data may indicate relatively high Kn rates during July and August, Kn rates for the same river may be negligible during the winter months, and even during "transitional periods" such as April, May and June, and September, October and November. Since construction and operating costs can increase markedly for ammonia removal at wastewater temperatures below 20°C, Kn rates should be based on site-specific data, particularly if ammonia removal is proposed for such periods.

Seasonal adjustments in Kn using the temperature correction relationships presented in Section 3.2.5 may be appropriate during months when wastewater temperatures are above 20°C. During months when temperatures fall below this level, nitrogen series data collected during these cooler periods may be necessary to select appropriate Kn rates to support cool weather nitrification.

3.2.3 Sediment Oxygen Demand (SOD) and Reaction Rate Effects

For over half the AT projects reviewed, separate SOD rates were not established. While SOD is not always significant, the potential impact of SOD on model results should be considered in all DO analyses. If SOD is not accounted for with a separate SOD rate in the model, the DO demand associated with SOD would be included in the Kd or Kn rates. The resultant overly high Kd and Kn rates could lead to overestimation of DO improvement from reduction of CBOD and ammonia, wherever significant SOD occurs.

For AT projects involving the use of SOD rates, the principal issues involved (1) the selection of appropriate stream locations for measuring representative SOD rates and (2) estimation of future SOD rates after controls are installed.

In some cases, the SOD rate was measured just below the discharge and then applied in the model to a large (e.g. 5 miles) stream segment. That measured SOD rate would not likely represent a large stream segment. Selection of representative locations for measuring SOD should account for the physical and hydrological characteristics of the river. Sampling location should be chosen to represent average solids deposition of the stream reach. If the streams geometry and velocity are nearly constant, measurement of SOD at only one or two locations may suffice to estimate a representative SOD rate. The estimation of a representative SOD is complex if the stream geometry and flow velocity vary or impoundments exist. In such cases, measurement of SOD in each nearly constant short segment may be necessary to estimate the composite SOD for the reach.

Another issue involves the selection of future SOD rates used in projection model runs. The primary mechanisms affecting future levels of SOD in streams and rivers include the rate at which existing deposits will be reduced after treatment and future deposition from continuing sources of settleable organics such as CSOs, algal biomass and other nonpoint sources.

Although future SOD rates cannot be measured, they will usually be less than existing SOD rates as a result of improved treatment. SOD rates determined from intensive surveys should be adjusted for projection model runs to reflect the controlled proportion of the settleable organic waste from all pollutant sources. Where significant SOD rates are estimated, a sensitivity analysis should be performed to ensure that AT is needed using conservative (i.e. resulting in less treatment) estimates of SOD.

3.2.4 Reaeration Rate (Ka)

All except two of the modeling analyses reviewed by the AT Task Force included empirically derived formulae to estimate Ka rather than direct stream measurements. In these two cases, measured values of Ka were adjusted with an empirical formula to reflect conditions evaluated in the calibration and projection model runs. The major issue evaluated in AT reviews for Ka was the over-reliance on the empirical formulae in determining Ka. Since model calibration often involved adjustment of rate constants other than Ka, modeling analyses relied heavily on empirical formulae. Use of such formulae with their inherent uncertainties can be avoided if Ka is measured and such measurements are obtained when stream flows most closely approximate existing and projected critical low flow conditions.

Generally, model documentation included limited or no analyses to determine whether the formula applied suited the waterbody (2) compare the Ka estimates derived from the formula applied to measurements taken on similar waterbodies, and (3) evaluate the uncertainty associated with the estimates of Ka. The variability of Ka estimates using these formulae is illustrated in Figure 3-2. As shown in this figure, the range of Ka values calculated with the O'Connor, Owens, Churchill and Tsivoglou equations was significant at all flow levels in the Muskinguum River. Although these formulae may not be equally applicable to this particular river, this figure illustrates the differences in the Ka values calculated using these four formulae. For some AT reviews, empirically derived Ka values resulted in a degree of uncertainty exceeding the degree of predicted water quality improvement from the proposed AT processes.

Low empirically derived Ka values for slow moving streams provided the primary support for some proposed advanced treatment projects. Since these formulae relate Ka to stream velocity, depth or elevation change, the effect of wind on reaeration is ignored. However, wind can be the major mechanism affecting reaeration in slower moving waters. Therefore, a certain degree of uncertainty is associated with Ka, estimates derived from these formulae, particularly for impoundments and slow moving streams.

Another key issue concerns the selection of a Ka equation developed for receiving water conditions similar to those being modeled. Table 3-1 summarizes the stream conditions for development of four reaeration equations (O'Connor, Owens, Churchill and Tsivoglou). The widest range of stream conditions were used to develope the O'Connor equation. O'Connor considered depths ranging from 1.0 to 30 feet, and velocities from 0.5 to 1.6 feet per second (fps). Owens and Churchill generally considered only fast moving and shallow streams with velocities ranging up to 5.0 fps, and depths from 0.4 to 11 feet. Tsivoglou developed his equation based on streams with velocities of 0.3 to 0.6 feet per second and depths of 2.0 to 3.0 feet. Even though the stream being modeled closely resembles those used for developing the applied Ka equation, some uncertainty will remain if site-specific measurements are lacking.

Since Ka values depend even more on site-specific conditions than other rate constants, stream monitoring programs should give a high priority to the measurment of Ka rates for DO analysis. Direct measurement of Ka can significantly reduce model uncertainty and clarify the need for AT. Tsivoglou and Neal make the following summary statement (using K2 to define the reaeration rate) in their paper entitled "Tracer Measurements of Reaeration: III, Predicting the Reaeration Capacity of Inland Streams"[20]:

> "It is emphasized that no predictive model for reaeration capacity is an entirely satisfactory substitute for direct tracer measurement of K_2 in all cases. In specific situations where relatively small differences in K_2 may lead to large changes in the costs of waste treatment and control and the greatest possible accuracy is needed, only direct field measurement of K_2 can provide positive assurance that the most economical solution has been determined."

3.2.5 Temperature Effects on Rate Constants

The reaction rates Kd, Kn, Ka and SOD are normally adjusted automatically in most models to different temperatures using the following general relationship:

$$K(t) = K(20)\ 0^{T-20}$$

where,

K(t) = reaction rate at specified temperature
K(20 = reaction rate at 20°C
0 = temperature adjustment coefficient
T = temperature being simulated

However, since the growth of nitrifying bacteria decreases or ceases at lower temperatures, the above temperature adjustment formula may not apply to stream temperatures below 20°C. Therefore, calibration data gathered when stream temperatures are less than 20°C may be needed to determine whether nitrification occurs at such temperatures. Site-specific seasonal data for other rate constants should also be collected if seasonal variations in these rates would affect the AT funding decision.

3.2.6 Background CBOD and Reaction Rate Effects

The background CBOD can significantly affect the DO deficit. As shown below, background CBOD concentrations from 1.0 mg/l to 3.0 mg/l could cause a DO deficit at the critical sag point ranging from 0.1 mg/l to 1.2 mg/l. Assuming a total available deficit of 2.4 mg/l, this range of background CBOD would account for between 5 and 50 percent of the available DO deficit. Background CBOD values may be greater if influenced by upstream swamps or other sources of organic material.

	Background CBOD5 (mg/l)**		
0*	1.0	2.0	3.0
5	0.39	0.78	1.17
10	0.21	0.42	0.63
15	0.15	0.30	0.45
25	0.09	0.18	0.27

* 0 = Ka/Kd

** Values indicate maximum DO deficit caused only by background $CBOD_5$. Calculations based on $CBOD_u/CBOD_5$ ratio of 3.0.

Because background CBOD can potentially use a significant portion of the available DO deficit, its effect on the DO analysis should be accounted for. The two major factors affecting the impacts of background CBOD on the DO system are the actual value of background CBOD itself and the rate at which this CBOD decays. Kd values are usually calibrated to correspond with total in-stream CBOD values below an outfall. Thus, calibrated Kd values would reflect a composite CBOD decay rate that includes decay of both background CBOD and effluent CBOD. This approach is acceptable where the decay rate of background CBOD approximates that for the effluent CBOD. This situation would generally apply where well treated effluents (i.e., high quality secondary or higher treatment levels) discharge into non-polluted or slightly polluted streams since the low decay rates of both background and effluent CBOD would obscure differences between effects of applying separate Kd rates to background and effluent CBOD.

On the other hand, where background CBOD could cause a significant DO deficit and calibrated Kd values are relatively high (for example, greater than 0.5 per day), use of a composite Kd rate to reflect decay of background and effluent CBOD may exaggerate the predicted effect of the background CBOD and correspondingly reduce the predicted effect of effluent CBOD. In this case, the actual Kd associated with background CBOD would be lower than the composite Kd, and the actual Kd associated with effluent CBOD may be higher than the composite Kd. A similar but reversed situation may occur if upstream waters are polluted due to marshes or swamps that contribute readily oxidizable organic matter, and a relatively high quality effluent is discharged.

Most computerized Streeter/Phelps models do not allow input of different Kd rates corresponding to the various sources of CBOD in the stream. However, with hand calculations, the effect on DO from using different Kd rates for the background and effluent CBOD can be evaluated. Where the calculated DO impact is significant, the background CBOD should be derived from site-specific data and considered in the sensitivity analysis.

3.3 Selecting Seasonal Conditions for Wasteload Allocations

3.3.1 Flow Selection

A Technical Guidance Manual[14] on this subject is available from EPA, Office of Water Regulations and Standards, Washington, D.C. The document recommends the use of a design stream flow with an averaging period corresponding to the exposure period established for the water quality criteria. A once in 10 year frequency will usually apply, although other frequencies may be appropriate in some cases as indicated by the manual.

3.3.2 Seasonal Water Quality Criteria

Few State water quality standards include separate criteria for the different seasons. Seasonal criteria are generally not needed in most waterbodies where a seasonal variation in temperature does not cause a change in resident fish species from a pollution tolerant group to a pollution intolerant group. On the other hand, use of a single water quality criterion to characterize multiple fish related uses can result in over-protection of the use and excessive treatment requirements during the seasons when the most sensitive species are not present.

3.3.3 Receiving Water Temperature Selection

The selection of the stream temperature for DO modeling is an important step since it affects both rate constants and the DO saturation. Review of AT modeling analyses suggested shortcomings in the selection of stream temperature and in the determination of whether the critical temperature and critical flow would occur simultaneously.

Mean daily temperatures of shallow streams may vary by as much as 5°C during a 30-day critical summer low flow period. A 5°C increase in water temperature, say from 22°C to 27°C, could decrease DO saturation from 8.8 mg/l to 8.1 mg/l. This could reduce the streams assimilative capacity by 25 percent; assuming a maximum DO deficit of about 4.0 mg/l, a background deficit of 1.0 mg/l, and a DO criterion of 5.0 mg/l. Therefore, the temperature used in the model should be carefully determined based on site-specific data.

Temperatures representative of the critical wasteload allocation period should be selected. The temperature selected for the critical low flow period should also correspond to the month when the streamflow and stream temperature would together result in the minimum assimilative capacity. A Technical Guidance Manual[14] on this subject is available from EPA, Office of Water Regulations and Standards, Washington, D.C. The manual recommends a simplified model run for each month of the critical season using mean monthly temperatures and flow. From this simplified analysis the critical condition of temperature and flow is selected for a more detailed modeling analysis.

3.3.4 Wasteload Selection

Since the expected performance of treatment facilities varies seasonally, modeling should account for seasonal variations of effluent loadings. Treatment plant performance data collected during intensive stream surveys is generally used in calibrating models. However, estimates of future pollutant loadings are needed for model projection runs to evaluate future water quality conditions with and without advanced treatment.

The projection model runs for many AT reviews did not account for seasonal variations in pollutant loadings. For example, in some cases, an effluent $CBOD_5$ concentration of 30 mg/l was selected as representing the expected performance of a secondary treatment facility during a critical summer period. Although 30 mg/l BOD_5 on a 30-day average basis defines secondary treatment for facilities other than ponds and trickling filters, this concentration is based on the expected poorer performance of a secondary treatment facility during the colder months. Secondary treatment facilities with effluent concentrations averaging 30 mg/l $CBOD_5$ in the winter can be expected to average 20 mg/l $CBOD_5$ or less during the summer. They may even achieve partial or full nitrification in the summer months in some cases. Therefore, the water quality improvement predicted from AT will be overestimated unless expected summer $CBOD_5$ concentrations for secondary treatment are applied for modeling critical summer low flow conditions.

3.3.5 Seasonal Wasteload Allocation Analyses

Perhaps the most important issue raised in the review of AT projects involved the frequent provision for AT (usually ammonia removal) on a year-round basis without a strong water quality based justification for AT during the non-summer seasons. As presented in Section 2.2 providing AT processes and capacities year-round can involve substantial capital and operating costs above that needed to achieve AT only during the summer. Therefore, modeling analyses should evaluate treatment needs for each season in which AT is required. Where an isolated treatment facility discharges into an effluent dominated stream during warmweather conditions, a desk-top analysis usually suffices to demonstrate that secondary treatment alone would not achieve the DO criterion. It is not so readily apparent, however, during which seasons or months AT is necessary. Since stream flows are usually greater and stream temperatures lower during non-summer months, secondary treatment may often meet DO criteria during much of the year. Therefore, the most challenging issue is often not if but when AT is required.

Several AT projects involving year-round requirements for ammonia removal were supported by a water quality analysis performed only for the critical summer low flow period. Another less obvious example of inadequate seasonal analyses, but significant in terms of the prospective AT cost, involved AT requirements extending into the spring and fall seasons without additional modeling analyses covering these periods. For example, a 2.0 mg/l limitation for ammonia from May 1 through October 31 could require 50 percent more aeration capacity than the same numerical limitation from June 15 through September 15. Modeling analyses with supporting data should support any AT requirements extending beyond the critical low flow season.

Stream assimilative capacity is generally lowest in late summer and early fall when streamflows are lowest and temperatures highest. Assimilative capacity generally increases in late fall and usually reaches a peak in late winter and early spring when flows are high and stream temperatures are still relatively cool. The following analysis illustrates the seasonal variability in assimilitive capacity. The anmalysis has been simplified to enhance key aspects of this issue and therefore should not be used to set seasonal AT requirements.

Stream flow and temperature data were collected for 96 streams in the upper Midwest, Northeast and Southeast U.S[21]. These data, applied to the basic first order decay relationship ($f= 0 \exp(0/1-0)$ where $0 = Ka/Kd$), were used to calculate the assimilative capacity (UOD loading capacity) of each stream during each month of the year. Then, the ratios of the assimilative capacity during the critical summer month to the assimilative capacity during each remaining month are calculated. These ratios show how the assimilative capacity changes between months. These assimilative capacity ratios are then summarized for the three U.S. hydrologic regions which include the 96 streams.

Figure 3-3 graphically illustrates the seasonal variation in stream flow for selected upper Midwestern streams using the ratio of the monthly 30/Q/10 flows to the minimum warm weather monthly 30/Q/10 flow. Figure 3-3 shows that monthly 30/Q/10 flows in spring and early summer are much greater than flows during late summer and early fall. For example, in March the maximum monthly 30/Q10 flow was over 14 times (i.e., ratio of 14) the minimum 30/Q/10 flow. These data also show that the seasonal changes in monthly 30/Q/10 flows vary greatly for different streams, even within the same hydrologic region. For example, the ratios in April and May range from 3 to 15. These wide ranges in stream flow ratios contribute to similarly wide ranges of assimilative capacity ratios as described below.

Figure 3-4 shows monthly stream temperatures for these 96 streams. These data show that temperatures are usually greater than 20°C during the summer. From late winter through the spring, temperatures increase from about 10°C or less to around 20°C. In April, temperatures range from about 10-20°C. This temperature range is critical because in-stream nitrification may be inhibited or negligible when stream temperatures are below 15°C. As with stream flow, these data also contribute to wide ranges of assimilative capacity ratios as described below.

The above data on both stream flow and temperature were applied together with the first order decay relationship "f" and the following equation to calculate assimilative capacity ratios for these 96 streams.

$$\frac{W(t)}{W(min)} = \frac{Dc(t)\ f(min)\ (QR(t) + Qe)}{Dc(min)\ f(t)\ (Q_R(min) + Q_e)}$$

where,

Wa(t) = allowable monthly effluent UOD loading
Wa(min) = allowable effluent UOD loading during critical warm weather low flow month
Dc(t) = allowable DO deficit during given month
Dc(min) = allowable DO deficit during critical warm-weather low flow month
$Q_R(t)$ = river flow during any month
$Q_R(min)$ = river flow during any warm-weather low flow month
Q_e = effluent flow

and f = 0 exp (0/1-0) where, 0 = Ka/Kd
f(t) =

The following assumptions were necessary to develop these ratios:

- A municipal discharge with a flow twice the minimum monthly 30/Q/10 flow is located on each of the streams evaluated.
- Reaction rates (Ka and Kd) are adjusted for temperature using the relationship presented in section 3.2.5.
- Reaction rates (Ka and Kd) are not adjusted as a function of stream flow.
- A 0 value of 10 was used for all calculations. A sensitivity analysis for different 0 values indicated that the assimilative capacity results would not be significantly affected by the 0 assumption.

Figure 3-5(a) shows assimilative capacity ratios calculated for the upper Midwestern streams. Allowable UOD loading ratios are relatively constant between June and August. In May, however, allowable UOD loadings range from about two to more than 20 times the minimum summer loading. Similarly in March and April, allowable UOD loadings range from about four to over 70 times the minimum allowable loading. These wide ranges of assimilative capacity ratios emphasize the need to conduct site-specific seasonal treatment analyses.

Seasonal calculations of assimilative capacity using simplified modeling procedures such as those described above can be used to determine months when treatment beyond secondary may be required and when intensive surveys should be performed. For example, assume that AT for ammonia removal is required during the critical summer low flow period and that the UOD of a secondary effluent is five times that of a nitrified effluent. Figure 3-5(a) indicates that AT would be required from June through September for all these streams since the allowable loadings shown for each of those months is always less than five times the minimum allowable load for AT (i.e., if the ratio is greater than five, secondary treatment would be adequate). Since, the ratios range from 3 to 25 in May and 4 to 75 in April, AT for these months would be needed for some streams but not others.

Assimilative capacity ratios calculated for streams in the Northeast, shown on Figure 3-5(b), indicate similar trends. However, the ratios calculated for Southeastern streams, shown in Figure 3.5(c) differ from those for the Midwest or the Northeast. For example, the assimilative capacity ratios are less than 5.0 year-round for many Southeastern streams. If AT were required in the summer for these streams, it might be required year-round because of small seasonal changes in assimilative capacity. Similarly, ratios are generally less than 5.0 for a longer summer period in the Southeast than in the upper Midwest and Northeast. Thus, for many streams in the Southeast ammonia removal could be required from May through November without a seasonal WLA analysis. However, any year-round ammonia removal requirements even in the Southeast should be supported by seasonal WLA analyses.

3.4 Pollutant Loadings

3.4.1 $CBOD_u$ Loading ($CBOD_u$:$CBOD_5$ Ratio)

Nearly all the AT projects reviewed involved a standard $CBOD_5$:$CBOD_u$ ratio to calculate model loadings and set effluent limitations. The ratios selected were either 1.5 or 2.0, with the same ratio applied to both secondary and advanced treatment. The $CBOD_u$:$CBOD_5$ ratio varies widely with the characteristics of the wastewater and the level of treatment. AT reviews consider two primary issues related to the $CBOD_u$:$CBOD_5$ ratio: (1) the variability of the ratio for different wastewaters especially where BOD rather than CBOD data are used and (2) the use of the same ratio to estimate loadings and set effluent limits for a range of treatment levels.

The Simplified Modeling Methodology[15] provides EPA guidance on how to select the appropriate $CBOD_u$:$CBOD_5$ ratio. However, this guidance contains limited information on the variability of this ratio. More recently, EPA reported[19] an average $CBOD_u$:$CBOD_5$ ratio of 2.84 and a standard deviation of 1.15 for 144 secondary and advanced treatment facilities. This variability for different wastewater and treatment levels indicates a need for site-specific measurements of $CBOD_u$.

The use of the same ratio in the calibration of pollutant loadings and in setting permit requirements for secondary and advanced treatment can result in overestimation of the water quality improvements from AT. The $CBOD_u:CBOD_5$ ratio depends on the bottle carbonaceous decay rate (K1) of the wastewater. As the level of treatment increases, the rate at which the CBOD oxidizes (K1) decreases. The following table illustrates this relationship.

CBODu/CBOD5	K1
1.5	0.23
1.8	0.15
2.5	0.10
3.1	0.075
3.9	0.050

Using a single ratio for both secondary and advanced treatment will result in a greater difference between the $CBOD_5$ values for secondary and advanced treatment, and thus overestimate the predicted DO improvement from AT. For example, assume a $CBOD_u:CBOD_5$ of 1.5, and an effluent $CBOD_5$ of 20 mg/l with secondary treatment and 10 mg/l with AT. The $CBOD_u$ would be 30 mg/l with secondary and 15 mg/l for AT; a difference of 15 mg/l. Alternatively, if the ratio is 1.5 for secondary and 2.0 for AT, the $CBOD_u$ would be 30 mg/l and 20 mg/l; a difference of only 10 mg/l. This analysis illustrates how the use of a single $CBOD_u:CBOD_5$ ratio can overestimate the DO improvement from CBOD removal.

The $CBOD_u:CBOD_5$ ratio is used to translating model output ($CBOD_u$) into effluent limits ($CBOD_5$). A higher ratio for AT than for secondary, as suggested above, would result in a more stringent effluent $CBOD_5$ limitation than use of the lower ratio for both. Nevertheless, the need for nitrification should be primarily based on the need to remove ammonia rather than CBOD because about 75 percent of the UOD removed by nitrification stems from ammonia reduction rather than $CBOD_5$ reduction. If a proposed AT process is needed solely to meet a $CBOD_5$ limitation using a given $CBOD_u$:$CBOD_5$ ratio, the critical DO at a slightly higher $CBOD_5$ limitation should be evaluated to determine if the designated uses can be achieved at the higher $CBOD_5$ limitation.

3.4.2 Calculation of NOD_u Loading

For some projects, the NOD_u loading input into the DO model was calculated by multiplying the allowable TKN concentration by 4.57. NOD_u is an estimate of the oxidizable nitrogen in the effluent. Organic nitrogen included in TKN must be hydrolized to ammonia before the nitrifying bacteria can oxidize the nitrogen. Thus, use of TKN instead of ammonia to calculate the NOD_u loading for the model can overestimate the NOD_u load if part of the organic nitrogen component of the TKN is not hydrolized in the stream above the critical sag point.

In well operated secondary and AT facilities, sufficient hydraulic detention time may be available to convert most of the readily oxidizable nitrogen to ammonia[22]. The stable organic nitrogen remaining in the effluent after treatment may not hydrolize to ammonia in the receiving water to create an oxygen demand within the critical reach. TKN can be used to calculate the NOD_u loadings if site-specific nitrogen series stream data identifies the rate at which organic nitrogen is hydrolized to ammonia in the stream, and thus impact the DO at the critical sag point. If these data are not available, ammonia should be used to calculate $NBOD_u$ loadings for calibration and projection model runs.

3.5 Reaction Rate Adjustment (Model Calibration)

Some AT issues concerned procedures used in model calibration rather than any particular model coefficient. Calibrating a water quality model basically involves two separate steps. First, individual reaction rates are estimated by fitting predicted pollutant concentrations to in-stream data (e.g., determining Kd by fitting CBOD decay) or through direct measurement (i.e., Ka or SOD measurements). Second, model rates are adjusted so that model results reproduce in-stream data. Since a given pollutant is affected more by some reaction rates than by others, each model rate must be determined individually.

Incorrect calibration of models was observed for some AT projects reviewed. For one project, Kd was adjusted until predicted DO corresponded to measured DO, rather than adjusting Kd to correspond to $CBOD_u$ data. SOD, not considered in the model, was measured at 0.36 $gm/m^2/day$. In this case, the effect of SOD was inadvertantly included in the Kd rate in order to fit the model output to observed data. In this case, the Kd rate used in the model was about 25 percent higher than the Kd estimated determined from CBOD data. Since the Kd used in the model exceeded its likely value, the DO increase resulting from $CBOD_5$ removal was overestimated.

After each reaction rate has been independently determined, the resultant model outputs should be compared with actual in-stream data. At this stage, model reaction rates may be adjusted to better fit the model results to the stream data. However, these adjustments should not exceed a predetermined range for each individual rate constant. For example, if $CBOD_5$ data showed that Kd could range from 0.25 to 0.30 per day (depending on how the slope of the CBOD decay curve was drawn), Kd should not be adjusted beyond this range. Often the most successful approach in calibrating a model is to set all rates other than Ka equal to their most reasonable value based on available data, and then to vary Ka within the range indicated by the applicable reaeration formulae or Ka measurements. If adjustments within this range of Ka do not produce a good fit of the data, then the other rate constants may be adjusted further within their range of uncertainty. If these adjustments still do not produce a good fit, the analyst should re-evaluate available data, the reaeration formula used and the receiving stream itself to identify factors that may be preventing model results from corresponding with actual in-stream data.

Since funding and time for stream sampling programs is often limited, it is essential that these programs be conducted in a cost-effective and timely manner. Highest priority should be given to data collection needed to reduce uncertainties with the greatest potential effect on the cost of treatment facilities. Therefore, before sampling programs are designed, the analyst should first conduct simplified modeling analyses using available data and literature values to identify the most significant areas of uncertainty. In addition, the analyst should also make preliminary cost estimates of various feasible treatment alternatives to assess effects of model uncertainties on treatment costs.

Even after calibrating a model, the analyst should recognize that uncertainties will remain when using the model to predict future conditions. Thus, the analyst should qualitatively consider the additional uncertainty of the model to predict future conditions and their possible effects on water quality and the justification for AT.

3.6 Permit Averaging Periods

A Technical Guidance Manual[14] on this subject is available from EPA, Office of Water Regulations and Standards, Washington, D.C. This manual recommends the use of the permit averaging period that is consistent with the averaging period of the water quality criteria being evaluated. Since chronic criteria are generally based on a 30 day average, results of wasteload allocations are presented as 30 day averages in the review of AT processes needed to meet chronic criteria. Averaging periods less than 30-days may be applied in cases where acute water quality criteria are evaluated.

A key issue raised in AT reviews involved permit averaging periods of less than 30 days. In some cases, permits were written for a 1-day or 7-day averaging periods to meet a chronic water quality criterion, either DO or ammonia, without analysis of acute effects for 1 to 7 days. Use of one-day or

7-day permit averaging periods often resulted in extra or larger AT facilities (e.g., aeration capacity or tertiary filtration) to meet effluent limitations during the shorter and therefore more critical periods.

Table 3-1 Stream Conditions For Developing Reaeration Relationships

	O'Connor	Tsivoglou	Owens	Churchill
H (ft)	1-30.0	2-3	0.4-11	2-11
U (fps)	0.5-1.6	0.3-0.6	0.1-5.0	1.8-5.0
Reaeration Equation[1]	$\frac{12.9U^{0.5}}{H^{1.5}}$	$0.054\ \frac{(\Delta h)}{(t)}$[2]	$\frac{21.7U^{0.67}}{H^{1.85}}$	$\frac{11.6U^{0.969}}{H^{1.673}}$

---

(1) Notation: U = velocity (ft/sec); H = average stream depth (ft.); (Δ h/t) = change in stream elevation (feet) per time (hours).

(2) Ka is given as 1/hour, at 25°C.

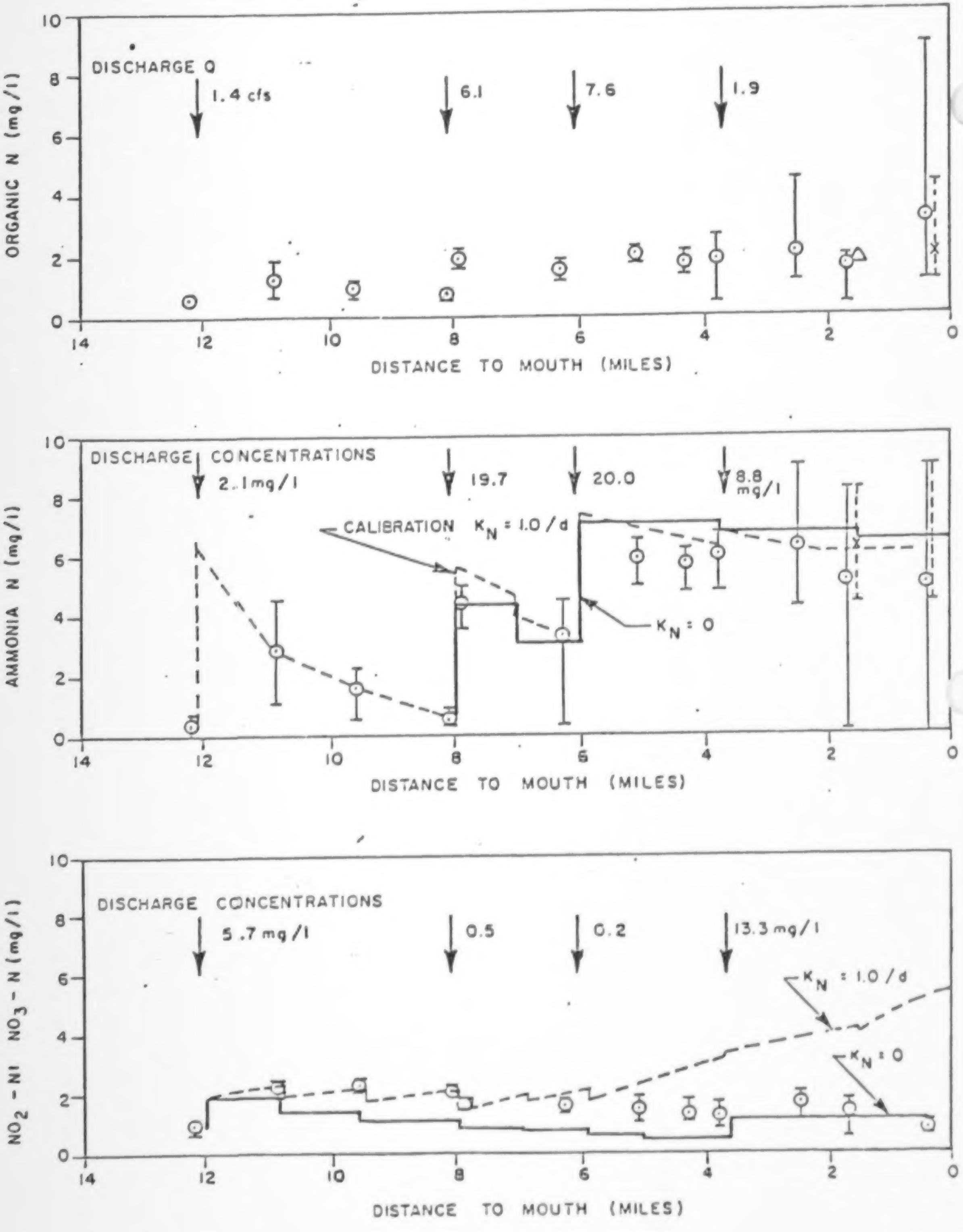


FIGURE 3-1 DETERMINATION OF K_N

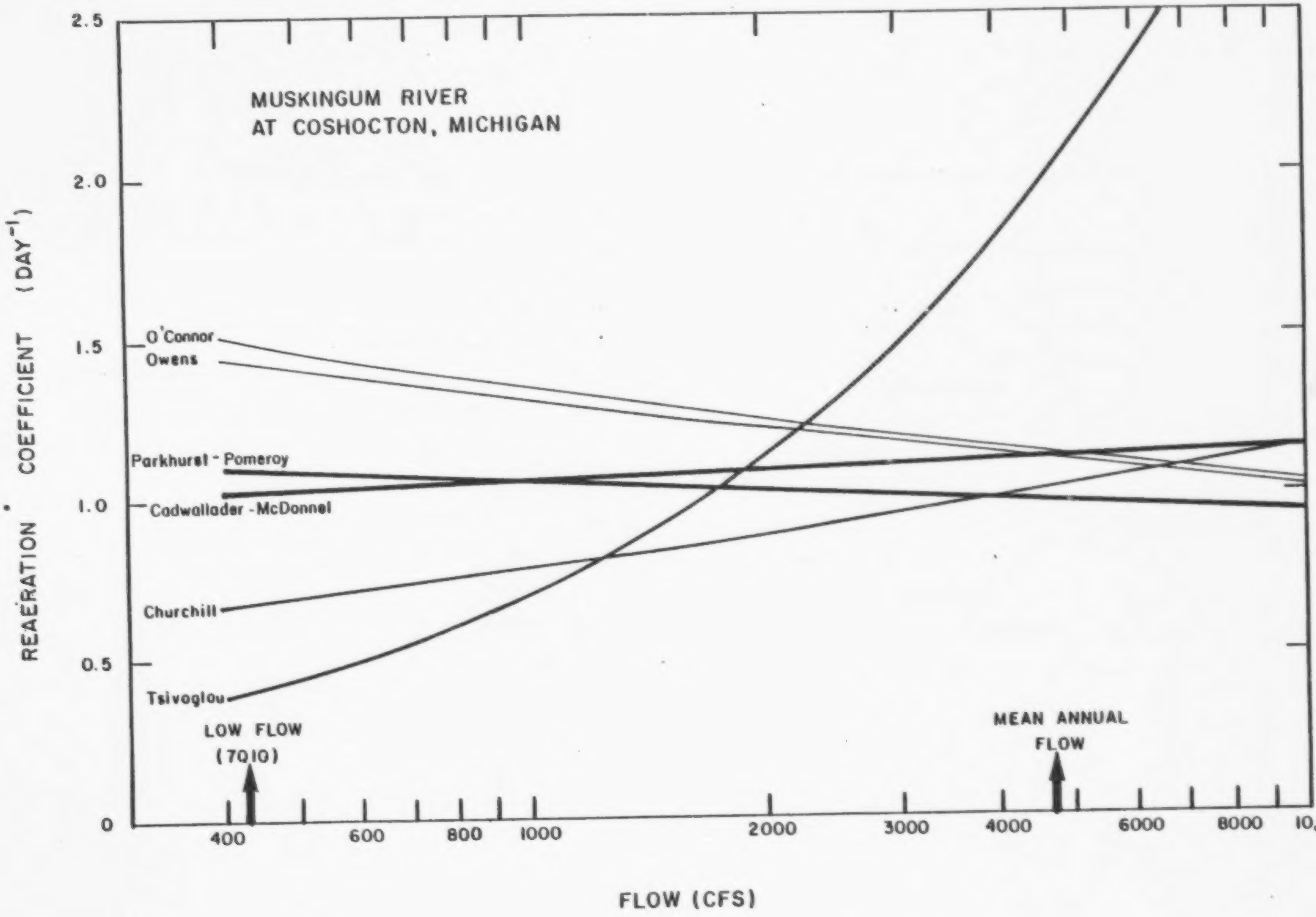


FIGURE 3-2 . STREAM REA ATION ACCORDING TO SIX FORMULAS

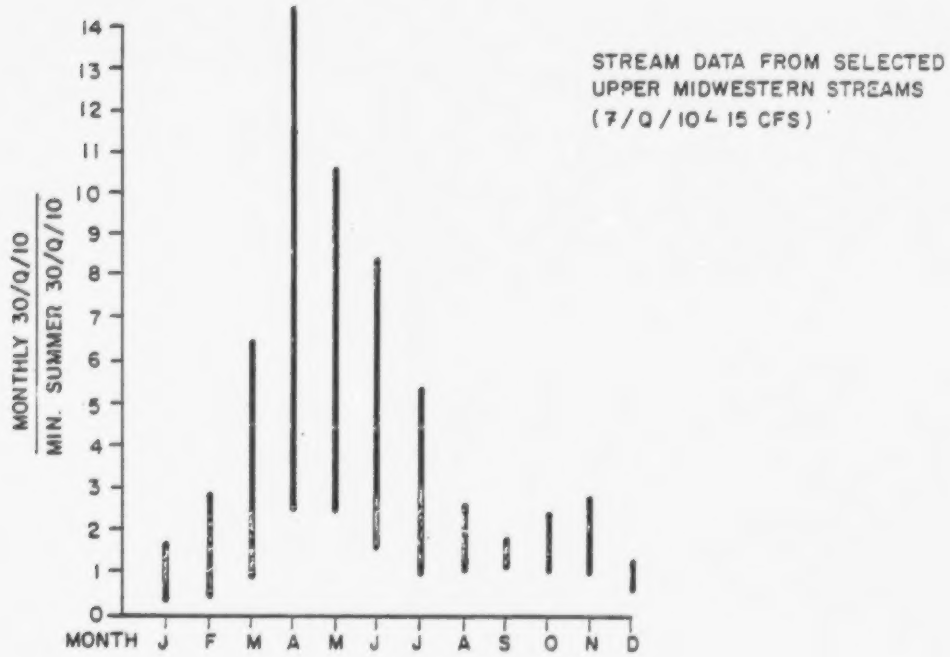


Figure 3-3

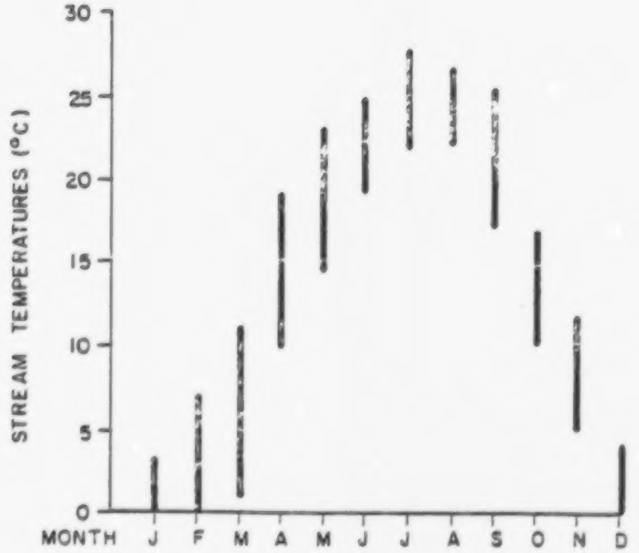


Figure 3-4

RANGES OF MONTHLY CHANGES IN STREAMFLOWS AND TEMPERATURES FOR SELECTED UPPER MIDWESTERN STREAMS

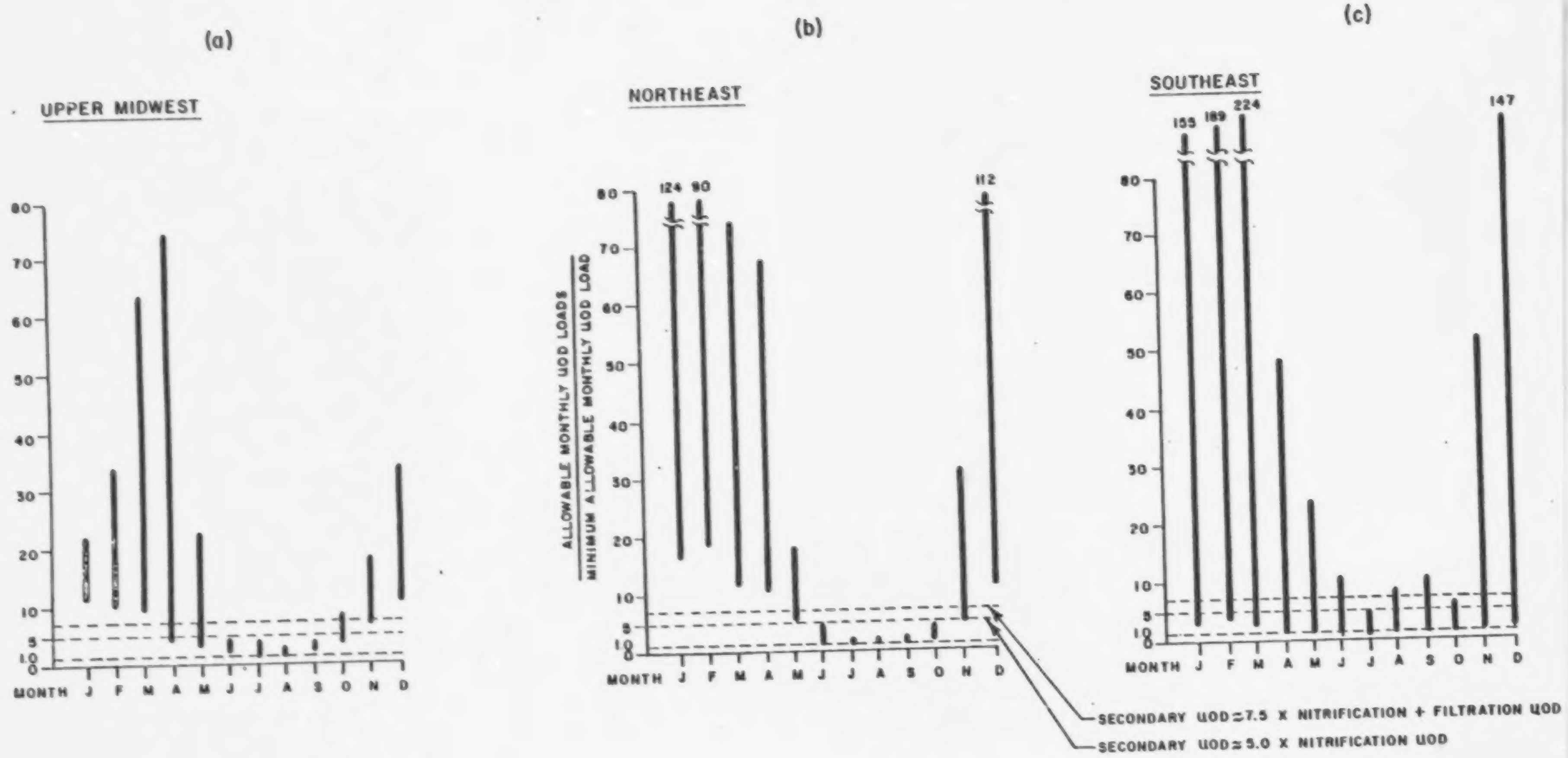


FIGURE 3-5 RANGES OF CHANGES IN RELATIVE ALLOWABLE UOD LOADINGS IN SELECTED STREAMS

4.0 Water Quality Issues -- Ammonia and Chlorine Toxicity

This section discusses issues raised during AT reviews of the water quality analyses used to establish treatment requirements to meet ammonia and chlorine criteria. The discussions in this section assume that the reader is familiar with the chemistry of ammonia and chlorine toxicity and the mass balance procedure for predicting in-stream concentrations and environmental effects. The information presented in this section should be used as a supplement to the Technical Guidance Manual Series[14] the Simplified Modeling Methodology[15,16] and the Water Quality Standards Handbook[17] available from EPA, Office of Water Regulations and Standards, Washington, D.C.

4.1 Ammonia Toxicity Analyses

4.1.1 Ammonia Criteria

Approximately twenty AT projects reviewed by the AT Task Force include nitrification proposed solely or principally to prevent ammonia toxicity. Chronic criteria used to establish the effluent limitations for the proposed AT projects ranged from 0.02 to 0.05 mg/l un-ionized ammonia at pH 8 and temperature of 20°C. These criteria were generally consistent with EPA ammonia criteria (1976)[18] and recent draft revisions.

One of the major criteria issues concerns the use of chronic ammonia criteria for an exposure period of less than 30 days. For one AT project, the standards included an un-ionized ammonia criterion of .04 mg/l "not to be exceeded at any time." The numerical un-ionized criterion closely approximates the EPA recommended chronic criteria range of 0.03 mg/l to 0.07 mg/l (depending on the fish species to be protected) for an exposure period of 30 days. Consequently, the daily maximum effluent limitation established to meet the .04 mg/l criterion on a daily basis is more stringent than that necessary to meet the EPA recommended acute toxicity criterion of .125 mg/l at a pH of 8 and a temperature of 20°C. This, in turn, required more treatment capacity for nitrification than necessary if the effluent limitations were applied on a 30-day average basis.

A final issue related to ammonia criteria involves application of National EPA Criteria to protect fisheries that do not contain members of the Salmonidae and Percidae families. The National Criteria represent the most sensitive salmonid family of fish. The latest chronic criteria information indicates that non-salmonids can be exposed to more than twice the concentration of ammonia as salmonids with no adverse effect. Thus, ammonia removal requirements as a basis for AT funding decisions should be based on the type of fishery (salmonids vs. non-salmonids) designated in the water quality standards for protection.

4.1.2 Stream Temperature Selection

The design stream temperature selected for ammonia toxicity analysis effects results since temperature, as well as pH, controls the percentage of total ammonia existing in the toxic un-ionized form. For example, if water temperature increases by 5°C, the percent of un-ionized ammonia at a pH of 7.5 would increase from 1.2 to 1.8, an increase of 50 percent. The analyst should therefore carefully select an appropriate design stream temperature in an ammonia toxicity analysis. A Technical Guidance Manual[14] on this subject is available from EPA, Office of Water Regulations and Standards, Washington, D.C.

4.1.3 pH Selection

A Technical Guidance Manual[14] on this subject is available from the EPA, Office of Water Regulations and Standards, Washington, D.C. The results of ammonia toxicity analyses is extremely sensitive to pH. For example, at a temperature of 25°C, a pH increase from 7.5 to 8.0 triples the percentage of the total ammonia in the un-ionized form (from 1.2 to 3.8 percent). As another example, a reduction in design stream pH from 7.8 to 7.5 could double the allowable effluent ammonia loading.

Ammonia toxicity analyses in support of AT projects were usually inadequately supported with in-stream pH data. In some cases, such analysis was based on less than 10 pH measurements for the season evaluated, with pH data lacking for some critically important coldwater months. While in some cases temperature and flow can be estimated reasonably well from limited information and data, pH estimates derived from limited data can be extremely variable and nonrepresentative of the conditions being analyzed. pH data can be collected easily with little expense in contrast to the expense of ammonia removal, particularly during non-summer periods. Accordingly, ammonia toxicity analyses should be supported with adequate pH data.

The design stream pH used in ammonia toxicity analyses should reflect the pH expected to occur after complete mixing of the upstream flows and the wastewater discharge. In most cases, the critical receiving water location will be the point of complete mixing. However, in some cases, a progressive increase of in-stream pH downstream from the discharge results from algal activity together with ammonia oxidation. In such cases the critical area for ammonia toxicity may be shifted downstream from the point of complete mixing.

The alkalinity of the receiving water and the effluent discharge may effect pH estimates below the mixing zones. Alkalinity is the ability of a water to absorb hydrogen ions (i.e., buffer pH changes). Therefore, the blended pH of a highly alkaline solution and a non-alkaline solution will approximate the pH of the highly alkaline solution. Simplified analytical procedures for determining blended pH of waters having different pH and alkalinity values are contained in Appendix A of the Simplified Modeling Methodology[15]. The example problem in this Appendix underscores the potential importance of considering the alkalinity of receiving waters and effluent discharges in determining the pH values below a discharge.

Accurate estimates of pH and alkalinity may be difficult to derive from available downstream data because future increases of wastewater flows and process modifications may alter pH and alkalinity values. Also, pH data obtained during normal or high stream flows may differ from pH values corresponding to design low flow conditions when effluent flow is a much larger proportion of streamflow. For these reasons, the best estimates of future downstream pH values may often depend on calculating downstream pH, accounting for upstream and effluent pH and alkalinity values.

4.1.4 Analysis Procedure

Many of the ammonia toxicity analyses presented to the AT Task Force applied a single value for temperature, pH and the un-ionized ammonia criterion in a simple mass balance calculation at the point of discharge under critical low flow conditions. This procedure may be misleading because of the uncertainty of the values selected in the analysis (pH, temperature, criteria) and the likelihood that these conditions will occur simultaneously. Where temperature and pH data are scarce or the data indicate variability, the results of the ammonia toxicity analysis should be presented as a range rather than an absolute value. This enables evaluation of the variability of the results and the likelihood that a given criterion will be violated.

To evaluate the sensitivity of an ammonia toxicity analysis to changes in flow, pH, temperature or the ammonia criterion, a table can be constructed in which one or more constituents are varied within a prescribed range. The following table presents a matrix of the maximum allowable 30-day average effluent ammonia concentrations as a function of a range of pH values and un-ionized ammonia criteria. This analysis assumes a wastewater discharge flow of 20 cfs and no upstream (background) ammonia load.

	Monthly 30Q10 Flow (cfs)	Avg. Temp (°C)	Max. allowable+ 30-day avg. effluent NH_3 concentration (mg/l)			
			pH7.5		pH7.8	
			(1)	(2)	(1)	(2)
Jan	32	3	37.2	74.4	18.7	37.4
Feb	97	3	83.6	167.2	42.1	84.2
Mar	290	5	193.7	387.4	99.2	198.4
Apr	150	10	70.5	141.0	36.5	73.0
May	60	15	23.6	47.2	12.0*	24.0
June	30	20	9.5*	19.0	5.2*	10.4*
July	19	26	5.8*	11.6*	2.5*	5.0*
Aug	10	26	4.5*	9.0*	2.0*	4.0*
Sept	10	20	5.7*	11.4*	3.1*	6.2*
Oct	11	15	9.1*	18.2	4.5*	9.0*
Nov	14	10	14.1*	28.2	7.3*	14.6*
Dec	30	5	31.2	62.4	16.0	32.0

+ Although ammonia criteria are not constant within the pH and temperature range covered by this anlaysis, they are assumed to be constant to simplify the analysis and highlight the sensitivity to pH and temperature.

* Values less than 15.0 mg/l; ammonia removal may be required.

(1) Based on a 0.05 mg/l un-ionized ammonia concentration

(2) Based on a 0.10 mg/l un-ionized ammonia concentration

This table indicates that the ammonia toxicity analysis can be highly variable within this range of pH values and ammonia criteria. (The table can also be constructed to reflect the variability of other factors such as temperature and flow). For example, a change in the pH from 7.8 to 7.5 approximately doubles the maximum allowable effluent ammonia concentration. A change in the criterion from 0.05 to 0.10 mg/l also approximately doubles the allowable ammonia concentration. From this matrix, the analyst can determine when ammonia removal is definitely needed to meet the prescribed criterion, the degree of violation that may occur during each month of the year and most importantly the sensitivity of the analysis to a range of values that reflect the variability of the data base.

Substantial data and careful review of the ammonia toxicity analysis is warranted whenever an ammonia toxicity analysis would provide the sole justification for ammonia removal. As discussed in Chapter 2.0, ammonia removal at wastewater temperature less that 20°C can significantly increase treatment costs. Therefore, if the ammonia toxicity analysis shows that ammonia removal is required during non-summer months, evaluation of the in put data (temperature, pH, flow, criteria) during these months should be done to determine the likelihood that these conditions will occur. If limited data are available, the additional nitrification capacity for cold weather should be deferred until additional data can be collected to substantiate the need for more stringent ammonia removal requirements.

4.2 Chlorine Toxicity Analyses

Disinfection processes and dechlorination are not considered advanced treatment and therefore do not require an AT review to receive Federal funding. However, AT projects that include chlorination without dechlorination are evaluated in an AT review to determine whether the in-stream concentration of chlorine will prevent attainment of the designated uses. Where chlorine concentrations are estimated to exceed established criteria, dechlorination is required to receive Federal funding for AT.

4.2.1 • Chlorine Criteria

More than 90 percent of the reviewed AT projects featured the use of chlorine as the disinfecting agent. Chlorine toxicity was evaluated for only about 10 percent of the projects, with dechlorination proposed for only a few of the projects reviewed. If AT is needed to improve DO or to prevent ammonia toxicity, chlorine toxicity should also be evaluated to ensure that chlorine does not prevent attainment of the fishery uses otherwise afforded by AT.

Chlorine toxicity was not evaluated for most AT projects principally because most State water quality standards do not include criteria for chlorine. Where chlorine toxicity was evaluated by the State, the criteria used were consistent with EPA chlorine criteria (1976)[18] and recent draft revisions.

4.2.2 Analysis Procedure

For AT projects featuring chlorination, the Task Force performed a mass-balance calculation of the expected downstream total residual chlorine (TRC) concentration and compared this concentration to the EPA criteria for chronic and acute toxicity. Where acute or chronic criteria would be violated, the degree of such excursions was considered to assess potential effects on the designated uses. Where the receiving water would not adequately dilute the effluent and chlorine criteria violations are therefore predicted, the designated fishery uses to be protected from oxygen demanding pollutants may not be achieved. In cases where the impact from chlorine is predicted to cause severe water quality degradation of the receiving water, dechlorination or the use of an alternative disinfection process has been recommended as a prerequisite for approving funding of AT.

The primary mechanism for severe chronic toxicity from chlorine is through the formation of monochloramines and dichloramines. Ammonia, in very small concentrations in a chlorinated effluent, combines with chlorine to form these chloramine compounds. Free chlorine and chloramines, measured together by the total residual chlorine (TRC) test procedure, are equally toxic to aquatic life. The evaluation of chlorine impacts includes the analysis of both acute and chronic effects of TRC in the receiving water.

The free chlorine component of TRC generally dissipates rapidly in the receiving water. However, chloramines are more stable forms of TRC and can persist in the receiving water and exert widespread impacts downstream of a discharge. Decay rates associated with free chlorine and chloramines are compiled in Table 4-1; reprinted from an EPA document entitled "Impacts of Wastewater Disinfection Practices in Coldwater Fisheries"[23]. As shown in this Table, the rate of decay of free chlorine and chloramine is extremely variable ranging almost three orders of magnitude from 0.7 to 178 per day. This variation prevents an accurate calculation of the areal extent of violations of chronic chlorine criteria without site-specific data.

The rate of decay of TRC in natural waters is dependent on several factors including but not limited to: oxidation reactions, combination with organic compounds, photochemical decay and decomposition. Since the decay rate is dependent on the concentration of organic compounds in the effluent and the receiving water, the decay rate may also change with different treatment levels. Therefore, an evaluation of TRC impact from an existing treatment facility may not be appropriate for predicting the impact from an upgraded AT facility. Intensive surveys on other AT facilities with similar wastewater and receiving water characteristics may be necessary to predict the impact of TRC with AT.

The following table is presented to illustrate the variability of the decay rate and its affect on the time and distance in the stream required to reduce the TRC concentration to below a chronic criterion of 10 ug/l. The calculation assumes a dilution ratio of 1:1 and an effluent TRC concentration of 2000 ug/l. The estimate of distance assumes a velocity of 0.50 feet per second (fps).

(1 day base e) Rate	Days/Minutes	(miles) Distances
0.05	9.2/13,248	75.4
0.07	6.6/ 9,504	54.1
1.00	4.6/ 6,624	37.7
1.50	3.1/ 4,464	25.4
2.00	2.3/ 3,312	18.8
5.00	0.9/ 1,325	7.4
10.00	0.5/ 662	4.1
20.00	0.2/ 331	1.6
50.00	0.1/ 132	0.8
100.00	0.05/ 66	0.4
200.00	0.02/ 33	0.2

As shown above, chlorine toxicity could negate the prospective fishery use enhancement from DO improvement afforded by the reduction of oxygen demanding pollutants. Due to the documented toxicity of chlorine in laboratories and field studies and the relatively small incremental cost for dechlorination compared to the cost of AT, the need for dechlorination should be checked for AT projects that propose the use of chlorine for disinfection. Wherever a mass balance calculation of chlorine concentration shows that acute or chronic criteria will be exceeded, dechlorination should be provided or an alternative disinfection process should be used unless intensive survey data shows that TRC decay will occur within the mixing zone.

TABLE 4-1 CHLORINE DECAY RATES

Chlorine Form	Decay rate, day^{-1}	Conditions	Reference
Free Chlorine (80% OCl$^-$)	50-178	Samples exposed to wind and sunlight.	Johnson (1978)
	4.6-18	Indoor samples exposed to UV light no stirring.	Johnson (1978)
	50-178	Initial rate for indoor samples due to easily oxidized compounds.	Johnson (1978)
Monochloramine	0.7-1.8	Indoors.	Johnson (1978)
	6.7-7.4	Outdoors samples exposed to sunlight and turbulence.	Johnson (1978)
Free chlorine in York River estuary (monochloramine formation suspected)	1.2-1.3		Johnson (1978) from Bender et al. (1975)
Monochloramine	1.3		Johnson (1978) from Bender et al. (1975)
Monochloramine in six Colorado Rivers	1.8-53	Light bottle field tests.	Heinemann et a (1981)
	5.0-104	Based on field data without dilution.	Heinemann et a (1981)
Total residual chlorine	20	Estimated from Sanguinsin and Morris (1975).	Haas (1981)

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Appendix A

Advanced Treatment Project Summaries

Handbook of
Advanced Treatment Review Issues

June 1984

Appendix A

Handbook of Advanced Treatment Review Issues

REGION I

SUMMARY OF ADVANCED TREATMENT REVIEWS

DATE SIGNED	PROJECT NAME FLOW	PROPOSED AT PROCESS	APPROVED AT PROCESSES	DEFERRED PROCESSES	CAPITAL COSTS ($)			
					TOTAL PROJECT	INCREMENTAL AT	TOTAL DEFERRED	TOTAL FUNDED
TOTAL PROJECTS REVIEWED BY HEADQUARTERS: 77 Projects					3374.7M	1503.5M	789.3M	2585.4M
10/82	BRISTOL, CT. 8.0 MGD	Regionalization involving expansion & upgrade of E. Bristol STP; seasonal nitrification	Approved as proposed	None	21.5M	4.1M	0	21.5M
6/11/80	KEENE, NH 8.0 MGD	New regional facility; seasonal nitrification	Approved as proposed	None	20.5M	3.9M	0	20.5M
5/23/80	GARDNER, MA 4.4 MGD	Two-stage nitrification; phosphorus removal	Nitrification facilities as proposed	Phosphorus removal	6.6M	2.8M	0	6.6M
10/15/79	MILFORD, MA 4.3 MGD	RBC nitrification facilities; phosphorus removal; tertiary filtration	Proceed as proposed; evaluate minor design changes, phosphorus loadings, chlorine toxicity, and ammonia limitations	None	10.2M	2.0M	0	10.2M
10/1/79	ROCHESTER, NH 3.9 MGD	Two-stage nitrification; tertiary filtration.	Two-stage nitrification	Tertiary filtration	7.7M	2.3M	1.3M	6.4M
9/28/79	SANFORD, ME 4.4 MGD	Higher CBOD removal; phosphorus removal	Proceed as proposed following public meeting	None	9.5M	4.0M	0	9.5M

SUMMARY OF ADVANCED TREATMENT REVIEWS

DATE SIGNED	PROJECT NAME FLOW	PROPOSED AT PROCESS	APPROVED AT PROCESSES	DEFERRED PROCESSES	CAPITAL COSTS ($)			
					TOTAL PROJECT	INCREMENTAL AT	TOTAL DEFERRED	TOTAL FUNDED
6/21/79	MERIDAN, CT 11.6 MGD	Two-stage nitrification; tertiary filtration; breakpoint chlorination	Two-stage nitrification	Tertiary filtration	28.8M	5.3M	2.0M	26.8M
6/28/79	LEOMINSTER, MA 8.0 MGD	Phosphorus removal; single-stage nitrification; tertiary filtration	Phosphorus removal; single-stage nitrification	Tertiary filtration	19.3M	2.8M	1.5M	17.8M
9/28/79	CRANSTON, RI 23.0 MGD	Two-stage nitrification; tertiary filtration	Approved as proposed; suggest staging treatment capacity	None	40.6M	6.8M	0	40.6M
6/22/79	BROCKTON, MA 18.0 MGD	Two-stage nitrification; phosphorus removal; tertiary filtration	Approved as proposed	None	26.6M	9.7M	0	26.6M
								Region II
12/6/83	ONEIDA COUNTY, NY 48.0 MGD	Seasonal nitrification	Seasonal nitrification limited to an avg. monthly flow of 34 MGD; dechlorination	One third of the aeration basin and sludge handling capacity; one clarifier	27.3M	3.1M	2.0M	25.3M
1/29/82	UPPER PASSAIC, NJ 16 MGD	Combine 11 facilities into 7 facilities; all 7 will provide nitrification and dechlorination; 6 provide tertiary filtration	Warm weather nitrification at 5 plants; dechlorination at all plants	Tertiary filtration at all plants; nitrification at two plants	25.7M	17.3M	8.5M	17.2M
1/29/82	NW BERGEN COUNTY, NJ 11.2 MGD	Year-round two-stage nitrification	Warm-weather nitrification; dechlorination	Cold weather nitrification capacity	21.2M	11.4M	8.5M	12.7M
11/30/79	MANASQUAN, NJ 8.1 MGD	High level of CBOD and NH_3 removal.	Facilities to treat 6.0 MGD to State limits	Facilities to treat additional 2.1 MGD to State limits	16.4M	1.2M	3.5M	12.9M

SUMMARY OF ADVANCED TREATMENT REVIEWS

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11/29/79	ROCKAWAY VALLEY, NJ 12.0 MGD	Single-stage nitrification; denitrification; tertiary filtration	Nitrification	Denitrification; tertiary filtration; diurnal equalization ponds.	38.6M	8.8M	2.9M	35.7M
9/28/79	ONEIDA, NY 2.5 MGD	Two-stage nitrification; tertiary filtration; phosphorus removal	Nitrification	Tertiary filtration	9.5M	2.8M	0.6M	8.9M
2/2/84	ITHACA, NY 10 MGD	Phosphorus removal discharge relocation	Approved as proposed	none	29.8M	6.2M	0	29.8M
								Region III
1/24/84	CHALFONT-NEW BRITAIN and DOYLESTOWN, PA (3 plants)	Year-round nitrification; tertiary filtration	Dechlorination, warm-weather nitrification, and phosphorus removal at all plants; summer denitrification at one plant	All tertiary filtration; summer denitrification at two plants	11.6M	5.1M	1.0M	10.6M
9/30/82	WAYNESBORO, VA 4.0 MGD	Two-stage nitrification and tertiary filtration	Warm-weather nitrification; dechlorination	Winter nitrification; tertiary filtration	3.9M	3.0M	1.3M	2.6M
1/6/81	LANCASTER, PA 29.7 MGD	Nitrification and phosphorus removal	Phosphorus removal; warm-weather low-flow nitrification	Cold-weather high flow nitrification	29.8M	19.8M	3.0M	26.8M
9/28/79	SOD RUN, MD 10.0 MGD	Two-stage nitrification; phosphorus removal; dechlorination.	Phosphorus removal; dechlorination.	Nitrification	21.6M	3.0M	2.7M	18.9M
9/28/79	BECKLEY, WV 3.5 MGD	Nitrification: tertiary filtration	None	Nitrification; tertiary filtration.	9.1M	2.2M	2.2M	6.9M

SUMMARY OF ADVANCED TREATMENT REVIEWS

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					TOTAL PROJECT	INCREMENTAL AT	TOTAL DEFERRED	TOTAL FUNDED
9/28/79	BALTIMORE, MD 170 MGD	Phosphorus removal	Approved as proposed	None	3.6M	3.6M	0	3.6M
6/29/79	PHILADELPHIA, PA NORTHEAST 250.0 MGD SOUTHEAST 100.0 MGD	High CBOD removals	Approved as proposed	None	254.4M 58.9M	2.5M 7.5M	0 0	254.4M 58.9M
							Region IV	
12/21/83	LAGRANGE, GA 8.2 MGD	High BOD removal; tertiary filtration	3/4 of the aeration basin-clarifier capacity	1/4 of the the aeration & clarifier capacity	18.2M	5.9M	4.1M	14.1M
9/7/83	LEXINGTON, KY 36.0 MGD	Nitrification; tertiary filtration	40% nitrification tankage	Tertiary filtration; 60% of nitrification tankage	80.7M	17.5M	15.8M	64.9M
1/17/83	LAKE TOHOPEKALIGA, FL COST-EFFECTIVE ANALYSIS 12 MGD	Land treatment; citrus irrigation+ rapid infiltration	Approved as proposed	None	96.6M	96.6M	0	96.6 M
5/3/82	ROME, GA 18.0 MGD	Nitrification; filtration	1/2 aeration tankage	1/2 aeration tankage; tertiary filtration	14.0M	5.5M	3.7M	10.3M
6/24/80	GREENSBORO, NC 20 MGD	Two-staged nitrification; tertiary filtration	Approved as proposed	None	39.5M	4.2M	0	39.5M
4/11/80	LAKE CITY, FL 3.0 MGD	Land application	Approved	None	5.5M	5.5M	0	5.5M
4/9/80	MACON, GA 2 PLANTS 28.0 MGD	Nitrification; filtration	Seasonal nitrification; filtration for one plant	Filtration for one plant	62.0M	17.5M	5.3M	56.7M

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SUMMARY OF ADVANCED TREATMENT REVIEWS

DATE SIGNED	PROJECT NAME FLOW	PROPOSED AT PROCESS	APPROVED AT PROCESSES	DEFERRED PROCESSES AT	CAPITAL COSTS ($)			
					TOTAL PROJECT	INCREMENTAL AT	TOTAL DEFERRED	TOTAL FUNDED
1/30/80	CHAPEL HILL, NC 8.0 MGD	Two-staged nitrification	Approved as proposed	None	11.6M	2.7M	0	11.6M
10/25/79	BURLINGTON, NC 9.5 MGD	Nitrification; filtration.	Approved as proposed	None	10.0M	7.0M	0	10.0M
10/25/79	HIGH POINT, NC 16.0 MGD	Nitrification; filtration.	Approved as proposed	None	19.3M	2.3M	0M	19.3M
10/12/79	NEWTON/ COVINGTON, GA 4.8 MGD	High CBOD removal; phosphorus removal	Phosphorus removal	High CBOD removal	4.8M	3.2M	2.5M	2.3M
9/28/79	LONDON, KY 4.0 MGD	Two-stage nitrification; tertiary filtration	Two-stage nitrification	Tertiary filtration	6.3M	2.1M	0.5M	5.8M
9/28/79	GREENSBORO, NC 16.0 MGD	Two-staged nitrification; filtration	Approved as proposed	None	7.4M	1.5M	0	7.4M
9/28/79	ANSON COUNTY, NC 3.5 MGD	Two-staged nitrification (TF-AS)	As proposed	N/a	9.4M	1.9M	0	9.4M
9/20/79	LEE CO./ FT. MYERS, FL 12.0 MGD	Secondary treatment to land application	As proposed except central plant will be 10 MGD rather than 12 MGD.	2 MGD capacity	48.1M	38.3M	2.5M	45.6M
6/22/79	ATLANTA, GA 41.0 MGD	Single-stage nitrification; w/regionalization; w/inter-basin transfer	Approved as proposed	None	208.0M	32.0M	0	208.0M

REGION V

SUMMARY OF ADVANCED TREATMENT REVIEWS

DATE SIGNED	PROJECT NAME FLOW	PROPOSED AT PROCESS	APPROVED AT PROCESSES	DEFERRED PROCESSES	CAPITAL COSTS ($)			
					TOTAL PROJECT	INCREMENTAL AT	TOTAL DEFERRED	TOTAL FUNDED
12/9/78	ROCHESTER, MN 14.0 MGD	2nd stage nitrification; phosphorus removal	Approved as proposed	None	36.7M	10.0M	0	36.7M
2/12/79	WALWORTH CO. WI	Nitrification	Nitrification	None	20.0M	4.0M	0	20.0M
9/28/79	PORTAGE, IN 3.5 MGD	2nd stage nitrification dual media filters	2nd stage nitrification	Dual media filters	7.5M	2.7M	3.3M	4.2M
9/28/79	ALBERT LEA, MN 12.5 MGD	2-stage nitrification; granular media	Approved as proposed	None	19.3M	3.4M	0	19.3M
9/28/79	RICHMOND, IN 14.0 MGD	Nitrification; tertiary filtration	Nitrification	Tertiary filtration	15.3M	6.9M	3.4M	11.9M
9/28/79	BOWLING GREEN, OH 8.0 MGD	Sand filtration; single-stage nitrification	Approved as proposed	None	17.7M	1.6M	0	17.7M
10/1/79	WAUPUN, WI 1.8 MGD	Nitrification; tertiary filtration	Nitrification	Tertiary filtration	5.2M	2.3M	2.0M	3.2M
10/1/79	VALPARAISO, IN 6.0 MGD	Nitrification; tertiary filtration	Nitrification	Tertiary filtration	10.0M	2.8M	.62M	9.38M

SUMMARY OF ADVANCED TREATMENT REVIEWS

DATE SIGNED	PROJECT NAME FLOW	PROPOSED AT PROCESS	APPROVED AT PROCESSES	DEFERRED PROCESSES	CAPITAL COSTS ($)			
					TOTAL PROJECT	INCREMENTAL AT	TOTAL DEFERRED	TOTAL FUNDED
10/29/79	FT. WAYNE, IN 60.0 MGD	Single stage nitrification dual media filters	Single-stage nitrification	Dual media filters	15.2M	15.2M	9.1M	6.1M
10/29/79	MIAMI CO, OH 12.0 MGD	2nd stage nitrification; sand filtration; phosphorus removal	2nd-stage nitrification	Sand filters; phosphorus removal	12.4M	1.4M	.6M	11.8M
10/29/79	MARION, IN 12.0 MGD	Nitrification; sand filtration	Nitrification	Sand filtration	13.4M	5.7M	1.5M	11.9M
10/29/79	MONTGOMERY CO, OH 13.4 MGD	Nitrification; phosphorus removal; sand filtration	Seasonal nitrification	Phosphorus removal; sand filtration	30.4M	6.4M	6.0M	24.0M
11/2/79	KALAMAZOO, MI 53.0 MGD	Filtration; activated carbon adsorption	Filtration; activated carbon adsorption	None	83.0M	8.0M	0	83.0M
11/2/79	ALLIANCE, OH 7.5 MGD	Single-stage nitrification; tertiary filtration; phosphorus removal	None	Single-stage nitrification; tertiary filtration; phosphorus removal	12.0M	6.1M	7.2M	4.8M
11/2/79	DAYTON, OH 72.0 MGD	2-stage nitrification phosphorus removal; mixed media filtration	2-stage nitrification	Phosphorus removal; mixed media filtration	82.0M	38.5M	11.8M	70.2M

SUMMARY OF ADVANCED TREATMENT REVIEWS

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					TOTAL PROJECT	INCREMENTAL AT	TOTAL DEFERRED	TOTAL FUNDED
11/5/79	MAHONING CO, BOARDMAN, OH 5.0 MGD	Single-stage nitrification; phosphorus removal; filtration	Single-stage nitrification	Phosphorus removal; filtration	7.7M	3.4M	1.3M	6.3M
11/5/79	COLUMBUS, OH 200.0 MGD	2nd stage nitrification;phosphorus removal; granular media filtration; breakpoint chlorination	2nd stage nitrification	Phosphorus removal; granular media filtration; breakpoint chlorination	112.4M	38.8M	22.0M	90.4M
11/30/79	SUMMIT CO, OH 2.4 MGD	Single stage nitrification phosphorus removal; micro-screening filtration; dechlorination		All processes deferred until completion of further studies	6.3M	2.3M	6.3M	0
1/30/80	KENT, OH 5.0 MGD	Single-stage nitrification; tertiary filtration (multi-media); phosphorus removal; dechlorination	Phosphorus removal; dechlorination	Single-stage nitrification multimedia filtration	5.9M	3.0M	1.9M	4.0M
2/19/80	WOODBRIDGE, IL 10.0 MGD	2nd stage nitrification; dual media filtration;	2nd-stage nitrification; dual media filtration;	None	30.8M	5.1M	0	30.8M
4/11/80	BATTLE CREEK, MI 27.0 MGD	1 or 2 stage nitrification; phosphorus removal; tertiary filters	Nitrification; phosphorus removal	Tertiary filters	42.0M	7.4M	5.4M	36.6M

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SUMMARY OF ADVANCED TREATMENT REVIEWS

DATE SIGNED	PROJECT NAME FLOW	PROPOSED AT PROCESS	APPROVED AT PROCESSES	DEFERRED PROCESSES	CAPITAL COSTS ($) TOTAL PROJECT	INCREMENTAL AT	TOTAL DEFERRED	TOTAL FUNDED
7/18/80	LAPEER, MI 4.0 MGD	Single-stage nitrification; filtration	Single-stage nitrification; filtration	None	24.0M	3.8M	0	24.0M
10/28/80	WARREN, OH 16.0 MGD	2-stage nitrification; dechlorination	Seasonal nitrification; dechlorination	None	21.8M	6.4M	0	21.8M
11/30/79	GALESBURG, IL 10.4 MGD	2-stage nitrification; dual media filtration	2-stage nitrification; Dual media filtration	None	29.5M	8.1M	0	29.5M
11/5/80	MADISON, WI 50.0 MGD	Single-stage nitrification.	Single-stage nitrification	None	25.5M	17.1M	0	25.5M
4/14/81	BEAVER DAM, WI 5.7 MGD	2-stage nitrification; sand filtration	None	All AT processes until control of non-point sources provided	13.9M	5.5M	5.5M	8.4 M
4/8/82	MINNEAPOLIS/ST. PAUL, MN 278.0 MGD	2-stage nitrification added to existing secondary treatment plant	Part of nitrification facilities	Remainder of facilities	129M	129M	82M	47M
1/4/82	CHICAGO, IL 3STP (facs) 2000 MGD	Tertiary filtration; year-round nitrification	Seasonal nitrification	Tertiary filtration; remainder of nitrification capacity	823M	664M	523M	300M

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10

REGION VI

SUMMARY OF ADVANCED TREATMENT REVIEWS

DATE SIGNED	PROJECT NAME FLOW	PROPOSED AT PROCESS	APPROVED AT PROCESSES	DEFERRED PROCESSES	CAPITAL COSTS ($)			
					TOTAL PROJECT	INCREMENTAL AT	TOTAL DEFERRED	TOTAL FUNDED
10/29/79	SAN ANTONIO, TX 154.0 MGD	2nd-stage nitrification; mixed media filtration	Seasonal nitrification	Mixed media filtration	109M	70M	15M	94M
								REGION VIII
6/29/79	SIOUX FALLS, SD 15.0 MGD	2nd-stage nitrification; multimedia filtration; breakpoint chlorination/dechlorination	All processes	None	49.1M	9.4M	0	49.1M
8/13/79	BOULDER, CO 17.5 MGD	2-stage nitrification; dechlorination	2-stage nitrification; dechlorination	None	18.8M	11.5M	0	18.8M
9/20/79	BOZEMAN, MT 7.0 MGD	Land application by infiltration (1/3 flow)	Land application by infiltration		9.0M	4.1M	0	9.0M
								Region IX
12/26/79	LASCD, CA (Long Beach) 25.0 MGD	Filtration	Filtration	None	8.6M	1.6M	0	8.6M
12/26/79	LASCD, CA (San Jose) 62.5 MGD	Filtration	Filtration	None	17.3M	2.5M	0	17.3M
3/27/80	LACS (Sepulveda) CA 40.0 MGD	Filtration	Filtration	None	69.2M	3.8M	0	69.2M
11/20/80	LAS VIRGENES, CA 4.5 MGD	Filtration; coagulation	None	Filtration; coagulation	35.0M	3.0M	3.0M	32.0M
11/20/80	CORONA, CA 8.0 MGD	2-stage nitrification; sand filtration; dechlorination; coagulation	2-stage nitrification; dechlorination	Sand filtration; coagulation	13.2M	3.8M	3.0M	10.2M